AD-A040187

1

DISTRIBUTION STATEMENT A

Best
Available
Copy

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi

CONTRACT REPORT [illegible]

STRENGTH AND STRESS-STRAIN BEHAVIOR OF ATCHAFALAYA LEVEE FOUNDATION SOILS

by

J. M. Duncan

AD-A040187

CONTRACT REPORT S-70-3

STRENGTH AND STRESS-STRAIN BEHAVIOR OF ATCHAFALAYA LEVEE FOUNDATION SOILS

A Report of an Investigation

by

J. M. Duncan

under

Contract No. DACW39-68-C-0078

with

U. S. Army Engineers Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi

January, 1970

College of Engineering
Office of Research Services
University of California
Berkeley, California
Report No. TE 70-1

FOREWORD

The investigation described in this report was performed under a modification to Contract No. DACW39-68-C-0078, "Behavior of Zoned Embankments and Embankments on Soft Foundations," between the U. S. Army Engineer Waterways Experiment Station (WES) and the University of California, Berkeley.

The objectives of this investigation were to compare the properties of foundation soils of the East Atchafalaya Basin Protection Levees as measured in undrained (Q) triaxial compression tests with those measured in plane strain and simple shear tests, and to study the effects of anisotropy and load duration on the strength and stress-strain behavior of these soils. The work was sponsored by the Mississippi River Commission, Office, Chief of Engineers, and administered by the Office of Research Services of the College of Engineering, University of California, Berkeley; the report was prepared by J. M. Duncan, Assistant Professor of Civil Engineering.

The contract was monitored by Mr. J. R. Compton, Chief, Embankment and Foundation Branch, under the general supervision of Mr. J. P. Sale, Chief, Soils Division, WES. Contracting Officer was COL Levi A. Brown, CE.

SUMMARY

The objective of this investigation was to relate the properties of the Atchafalaya levee foundation soils measured in triaxial tests to those measured in plane strain and simple shear tests, and to study the effects of anisotropy and load duration on the strength and stress-strain behavior of these soils.

The strength and stress-strain characteristics determined in unconsolidated-undrained (Q) triaxial and plane strain tests were found to be essentially identical. The behavior of these soils in unconsolidated-undrained simple shear tests was found to be consistent with the results of triaxial and plane strain tests provided that proper account was taken of the initial stresses within the specimens and the anisotropic strength characteristics of the clay.

The undrained strength of the clay was found to vary depending on the orientation of the test specimens. Horizontal specimens were strongest, and those trimmed so that failure occurred on a horizontal plane were weakest. Although no creep strength loss was observed in the creep tests conducted, it was found that test specimens deformed continually under sustained load.

The Atchafalaya levees are being designed by the U. S. Army Engineer District, New Orleans, under the general supervision of the Mississippi River Commission. The investigation described in this report was sponsored by the Mississippi River Commission.

TABLE OF CONTENTS

LIST OF FIGURES

Figure
Number

LIST OF TABLES

LIST OF SYMBOLS

English Letters

c	cohesion intercept
E_i	initial tangent modulus
E_t	tangent modulus
f	subscript denoting failure
K	modulus number
K_o	at-rest earth pressure coefficient
n	modulus exponent
p	consolidation pressure
p_a	atmospheric pressure
p_o'	effective overburden pressure
R_f	failure ratio
S_u	undrained shear strength
x,y,z	coordinate axes

Greek Letters

α	a factor representing the ratio of the initial effective stresses in a laboratory specimen to those in-situ
β	angle between specimen axis and horizontal
γ	shear specimen
Δ	indicates a change in the appended quantity
ε	normal strain
ν	Poisson's ratio
σ	normal stress
σ'	effective normal stress
σ_1	major principal stress

σ_3	minor principal stress
$(\sigma_1 - \sigma_3)_f$	stress difference at failure, compressive strength
$(\sigma_1 - \sigma_3)_{ult}$	asymptotic value of stress difference
τ	shear stress
τ_{max}	maximum shear stress
ϕ	angle of internal friction

INTRODUCTION

Extremely poor foundation conditions in the southern half of the Atchafalaya Floodway in south-central Louisiana have severely hampered progress in constructing levees. The soils in this area, predominantly soft organic clays, are subject to large amounts of settlement under fills of even moderate height. Stage construction procedures have been employed in an attempt to raise the levees to their desired heights, 20 ft to 25 ft above the original grade, but even very slow construction over a period of 30 years has not forestalled large settlements. Over many long stretches the settlements have been 10 ft to 15 ft, and in some cases as much as 20 ft (Kaufman and Weaver, 1967).

Instrumented test sections have been built to obtain information concerning the mechanism of settlement, and to determine how settlements and movements were affected by the use of flat side slopes and stability berms. These studies showed that the settlements were accompanied by lateral movements and that the amount of lateral movement was greatest for sections having the lowest factor of safety with regard to shear failure. These movements, which continued over a considerable period, were found to be associated with continued shear deformation, the movements under the flanks of the levees being primarily horizontal (Kaufman and Weaver, 1967).

A number of studies have been and are being conducted to relate the observations from the instrumented sections to the properties of the foundation soils determined in laboratory tests. Limit equilibrium stability analyses have been used to develop information concerning the relationship between factor of safety and amount of movement (Kaufman

and Weaver, 1967), and finite element analyses are being conducted by the Waterways Experiment Station to determine if the observed movements can be predicted using stress-strain characteristics measured in laboratory tests. Toward this end, the Waterways Experiment Station is conducting a program of shear tests on foundation soils to define their time-deformation characteristics. The study described in this report, conducted to supplement the data obtained by the Waterways Experiment Station, was performed to relate the characteristics being determined by triaxial tests to those determined under conditions of plane strain and simple shear.

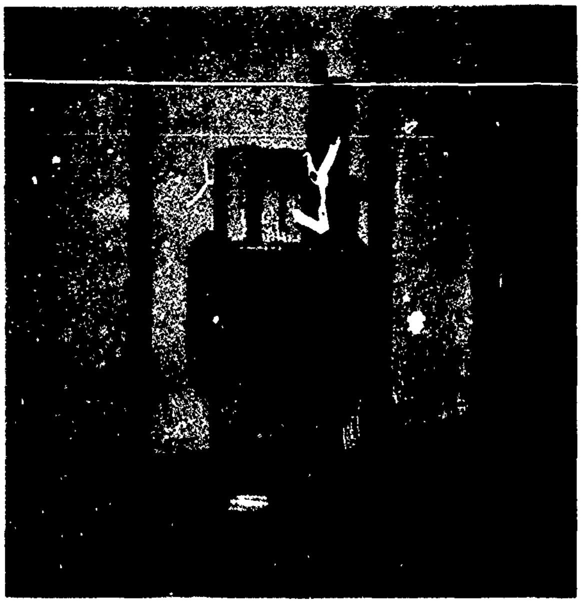
The Atchafalaya levees are being designed by the U. S. Army Engineer District, New Orleans, under the general supervision of the Mississippi River Commission. The investigation described in this report was sponsored by the Mississippi River Commission.

TYPES OF TESTS

During this study, four types of unconsolidated-undrained (Q) tests were conducted on high-quality undisturbed samples. These tests, illustrated in Fig. 1, included triaxial tests, plane strain tests, Geonor simple shear tests, and Cambridge simple shear tests. The procedures followed in these tests are described in the Appendix.

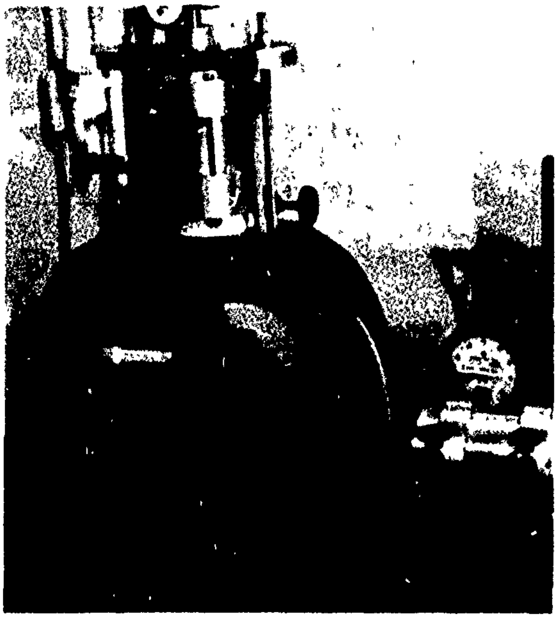
Triaxial tests were conducted to establish a standard for comparison with the results of the other types of tests, and to study the effects of confining pressure, cap and base restraint, anisotropy, and time-dependent deformations. These tests were conducted on 1.4 in diameter specimens which varied in length from 3.3 in to 3.5 in, using the equipment and procedures employed in previous studies of shear strength conducted for the Corps of Engineers (Duncan and Seed, 1965a; Duncan and Seed, 1965b). A photograph of a triaxial specimen on the base of the triaxial chamber is shown at the upper left in Fig. 1.

Triaxial



Plane Strain

Geonor Simple Shear



Cambridge Simple Shear

Fig. 1 PHOTOGRAPHS OF TEST SPECIMENS AND APPARATUS.

Plane strain tests were conducted on specimens 1.1 in wide, 2.8 in long, and 2.8 in high using the equipment shown at the upper right of Fig. 1. The equipment and procedures employed in these tests were the same as that used in a previous study of the effect of cap and base restraint (Duncan, Seed and Dunlop, 1966). During the study described in this report, tests were conducted on vertical specimens using normal caps and bases, consisting of polished lucite, without filter stones.

Geonor simple shear tests were conducted using the apparatus shown at the lower left in Fig. 1, which was manufactured by the Norwegian Geotechnical Institute. The principles and use of the apparatus have been described by Kjellman (1951) and Bjerrum and Landva (1966). The test specimen, 3.14 in diameter and 0.79 in high, is enclosed in a reinforced rubber membrane which restricts bulging deformation of the specimen while permitting the top to move horizontally with respect to the bottom as shown in Fig. 2. The specimen cap and base are mounted on low-friction guides which allow vertical movement of the cap and horizontal movement of the base while the two retain their parallel alignment. Specimens are first subjected to vertical (axial) load, and then to sufficient horizontal shearing load to cause failure. The procedures employed in trimming test specimens, assembling the apparatus and conducting tests were developed by the Norwegian Geotechnical Institute.

Cambridge simple shear tests were conducted using the apparatus shown at the lower right in Fig. 1, which was constructed at the University of California at Berkeley following a design developed by Roscoe (1953). The use of this apparatus for testing clays has been discussed by Dunlop, Duncan, and Seed (1968). The test specimens, 0.80 in high and 2.30 in square in plan dimension, are sealed inside a rubber membrane and surrounded by heavy metal walls as shown in Fig. 2. As in the Geonor apparatus, the

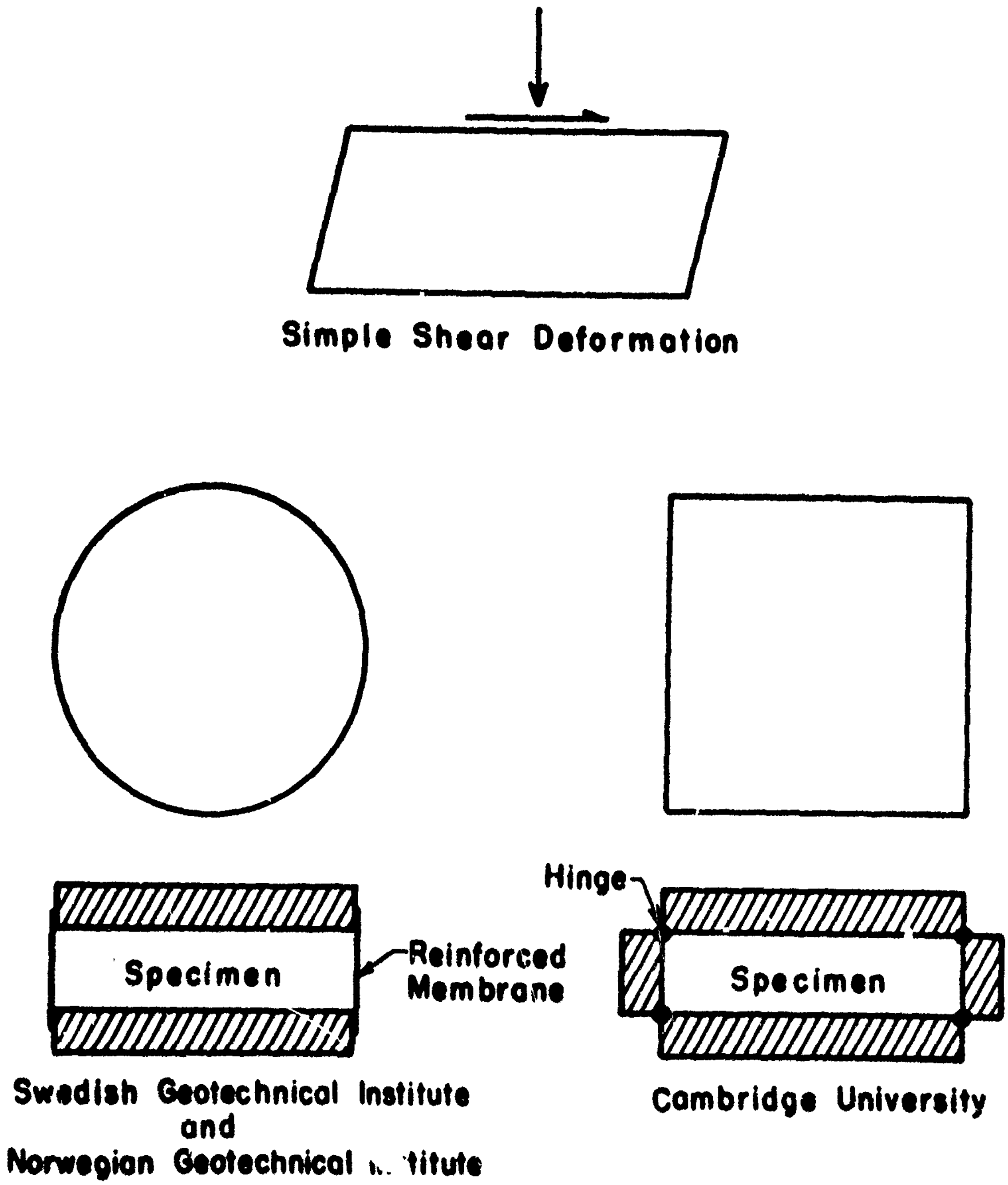


Fig. 2 SIMPLE SHEAR DEFORMATION AND SIMPLE SHEAR APPARATUS.

cap and base are mounted on low friction guides which allow vertical movement of the cap and horizontal movement of the base while the two retain their parallel alignment. The metal sides, which enclose the specimen in its membrane, are hinged so that the specimen can undergo shear deformation when subjected to horizontal shear loads. The procedures employed in these tests are essentially the same as those used previously by Dunlop, Duncan, and Seed (1968).

The stress conditions in simple shear test specimens are not uniform, because shear stresses are applied to the upper and lower surfaces of the specimen without complementary shear stresses on the vertical specimen surfaces. Duncan and Dunlop (1969) have shown, however, that reasonable interpretations of the tests may be made if the stresses within the specimens are assumed to be uniform. Because shear stresses are induced in simple shear specimens by application of the axial load, it is necessary to consider the combined state of stress induced by both the axial load and the shear load in order to determine the shear strength of the soil from the results of simple shear tests. This aspect of the test interpretation is discussed in detail in a later section of this report.

SOIL SAMPLES

The soil tested in this study is a dark grey, highly plastic clay of medium consistency from the foundation of levee Test Section III. It contains occasional bands of lighter grey silt about one-fourth inch thick, and a few roots one-fourth to one-half inch in diameter, but on the whole appears to be quite uniform. The samples tested were obtained from two borings on the landside of Test Section III (WES, 1969) at a sufficient distance from the levee so the samples were not affected by past construction. Twelve sections

of sample, each about one foot long, were obtained from these borings between depths of 67 ft and 75 ft beneath the ground surface. These sections, about 4.5 inches in diameter, were shipped to the University of California Soil Mechanics Laboratory sealed inside approximately one-half inch of wax. The wax was removed and the sections were cut to length for testing using a power band saw. Specimens were trimmed from these lengths using a wire saw. The study included 39 triaxial tests, 14 plane strain tests, 9 Geonor simple shear tests and 5 Cambridge simple shear tests. The locations of the test specimens are listed in Table 1.

TEST RESULTS

Triaxial Tests on Vertical Specimens

A number of triaxial tests were conducted for the purpose of comparing with the results of plane strain and simple shear tests, to determine the correspondence between the results of these tests for the Atchafalaya levee clay. Additional triaxial tests were also conducted to study the effects of confining pressure, cap and base restraint, anisotropy, and time-dependent deformations. As mentioned previously, all of these tests were conducted under unconsoidated-undrained (Q) test conditions.

The values of compressive strength measured in stress-controlled triaxial tests conducted using three different values of confining pressure are shown in Fig. 3. It may be noted that the compressive strength is not affected by the value of confining pressure, and that the scatter in the measured strength values is quite small. The range of stress-strain curves for these tests, shown in Fig. 4, is also quite small, indicating that the clay is very uniform with regard to stress-strain characteristics as well

Table 1. LOCATIONS OF TEST SPECIMENS

Boring	Sample	Sub-Sample	Depth (feet)	Section Length (inches)	Tests	Specimen Numbers
95UES	18	3	68.1	4.0	2 Triaxial 1 Plane Strain	18BT1,18BT2(contained root) 18BPS1
			to	3.0	1 Geonor	18BG2(contained root)
			69.1	5.0	1 Geonor 2 Cambridge	18BG1(sheared at surface) 18BC1,18BC2
		C	69.1	4.0	2 Plane Strain 1 Cambridge	18CPS2(disturbed),18CPS3 18CC3(disturbed)
			to	1.5	1 Geonor	18CG4a(sheared at surface)
				2.5	1 Geonor	18CG3
			70.3	4.0	4 Triaxial	18CT3,18CT4,18CT5,18CT6
	19	B	71.7	2.5	2 Triaxial	19BT13,19BT14
				3.5	2 Geonor	19BG4,19BG5(shear at sur.)
			to	2.0	2 Cambridge	19BC6,19BC7
				3.5	2 Geonor	19BG6,19BG7(disturbed)
			73.0	2.5	1 Geonor	19BG8
		C	73.0	3.5	4 Triaxial	19CT7,19CT8,19CT9,19CT10
			to	3.0	2 Plane Strain	19CPS4,19CPS5
				3.5	2 Triaxial	19CT15,19CT16
			74.2	4.0	2 Triaxial	19CT11,19CT12
96UES	18	B	67.9	4.0	4 Triaxial	18BT22,18BT23,18BT24,18BT25
			to	4.0	2 Triaxial	18BT20,18BT21
				3.0	1 Triaxial	18BT17
			68.9(?)	2.0	2 Triaxial	18BT18,18BT19
		C	68.9(?)	4.0	3 Triaxial	18CT27,18CT28,18CT29
			to	3.5	1 Triaxial	Specimens Disturbed
			70.2	4.0	2 Triaxial	Specimens Disturbed
	19	B	71.9	4.0	4 Triaxial	Specimens Dried Out?
			to	4.0	3 Triaxial	19BT37,19BT38,19BT39
			73.0	4.0	3 Plane Strain	Specimens Dried Out?
		C	73.0	3.5	2 Plane Strain	19CPS10,19CPS11(disturbed)
				3.1	2 Plane Strain	19CPS12,19CPS13
			to	3.1	2 Plane Strain	19CPS14,19CPS15
			74.2	3.5	Not Tested	

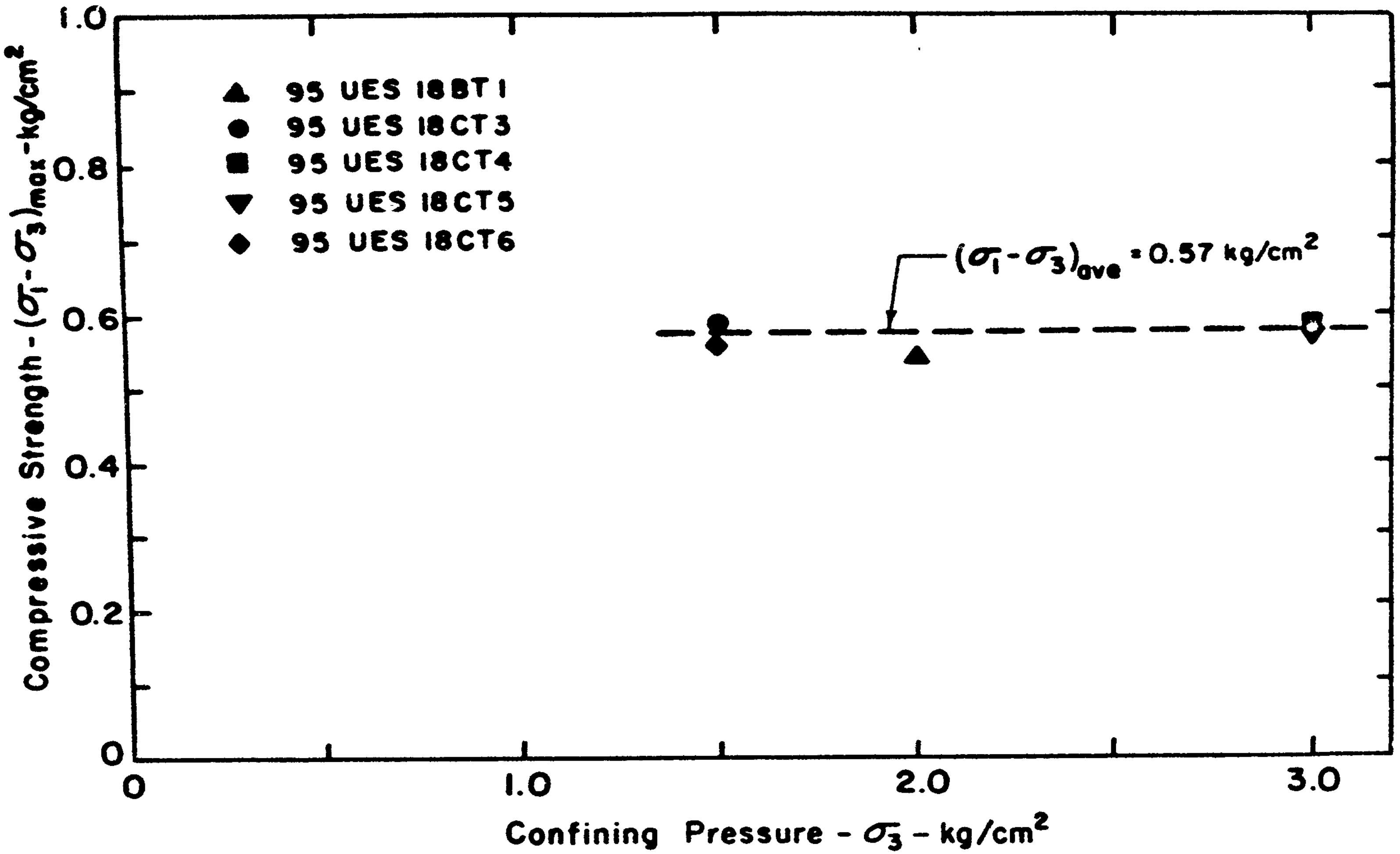


FIG. 3 VARIATIONS OF TRIAXIAL COMPRESSIVE STRENGTH WITH CONFINING PRESSURE.

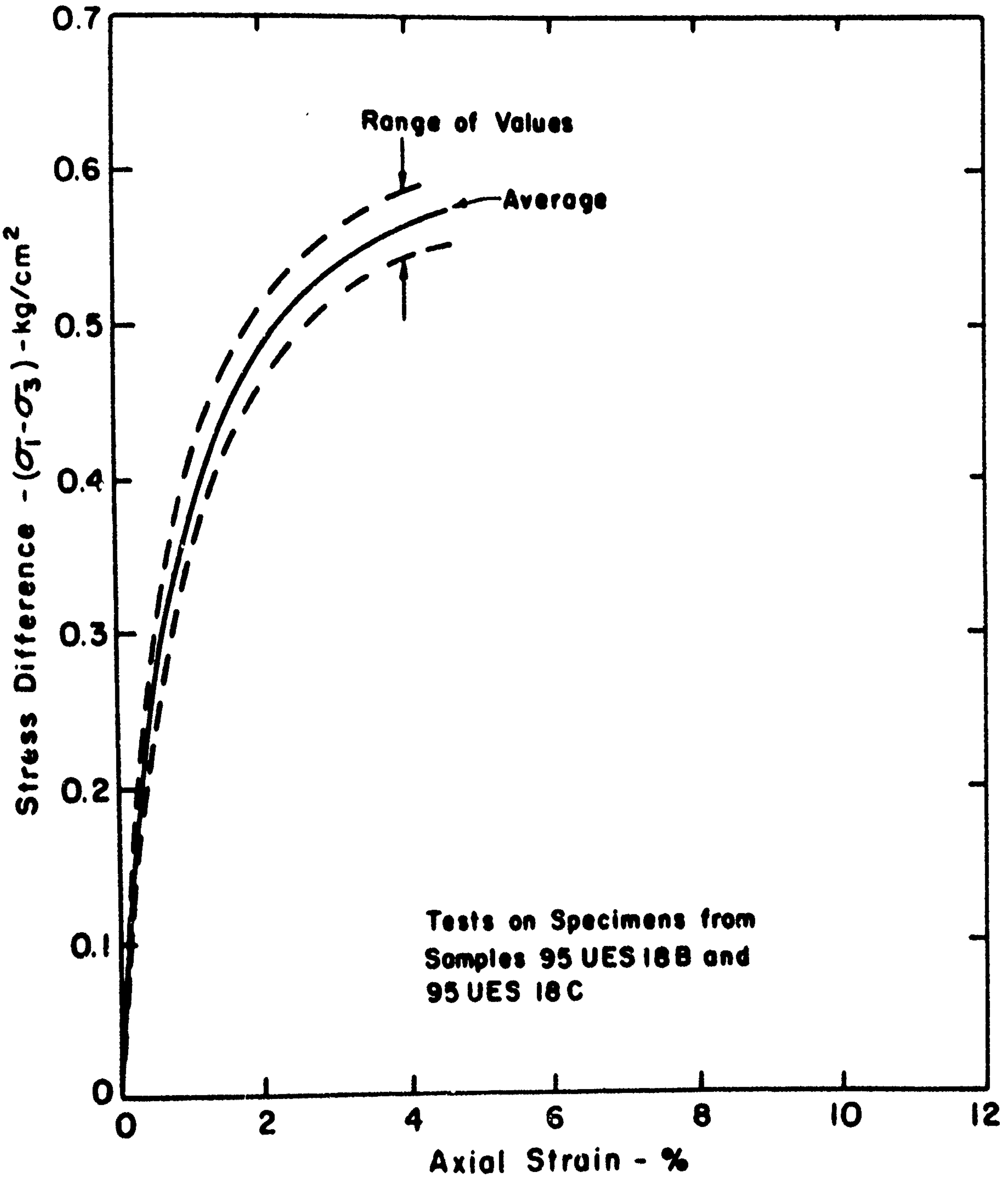


Fig. 4 RANGE OF STRESS-STRAIN CURVES FOR STRESS-CONTROLLED TRIAXIAL TESTS ON VERTICAL SPECIMENS.

as undrained strength. The fact that the clay is so uniform made it possible to expand the breadth of the testing program to include the effects of a larger number of factors than would have been possible had it been necessary to perform a large number of each type of test to achieve reliable average results. The results of the tests discussed previously are summarized in Table 2. It may be noted that the greatest deviation from the average measured value of compressive strength is only 0.02 kg/cm^2, or about 4% of the measured values. The measured value of undrained shear strength, $S_u = 1/2(\sigma_1 - \sigma_3)_f = 0.28$ kg/cm^2, is quite small in comparison with the effective overburden pressure at the depth from which the samples were obtained. At a depth of 70 ft beneath ground surface, data presented by Kaufman and Weaver (1967) indicate that the effective overburden pressure is approximately 1.2 kg/cm^2. Thus the ratio of undrained strength to effective overburden pressure, S_u/p, is about 0.23, and the rate of increase of undrained strength with depth in the clay is about 8 psf per ft of depth.

Stress-strain curves for additional triaxial tests, conducted using strain-control test procedures, are shown in Fig. 5. Using these procedures, it was possible to determine the stress-strain characteristics beyond the point where the peak strength is mobilized. It may be noted that the peak strength of this clay is mobilized at a very small value of strain (2%), and that the shear resistance falls off rapidly with additional strain beyond the peak. Kaufman and Weaver (1967) have pointed out that this type of brittle stress-strain behavior is typical of the Atchafalaya organic clays from shallower depths as well. Because brittle stress-strain behavior may lead to progressive failure, it is possible that this aspect of the stress-strain behavior has played a part in the large movements of the Atchafalaya levees.

Table 2. RESULTS OF STRESS-CONTROLLED TRIAXIAL TESTS ON VERTICAL SPECIMENS

Sample	Water Content	Confining Pressure kg/cm^2	$(\sigma_1 - \sigma_3)_f$ kg/cm^2	ε_f Strain at Failure	Time to Failure Minutes
95UES18BT1	72.4%	2.0	0.538	3%	11
95UES18CT3	73.9%	1.5	0.582	4%	12
95UES18CT4	74.4%	3.0	0.576	5%	12
95UES18CT5	73.6%	3.0	0.558	3.5%	12
95UES18CT6	74.0%	1.5	0.552	5%	11.5
Average	73.7%	---	0.56	4.1%	11.7

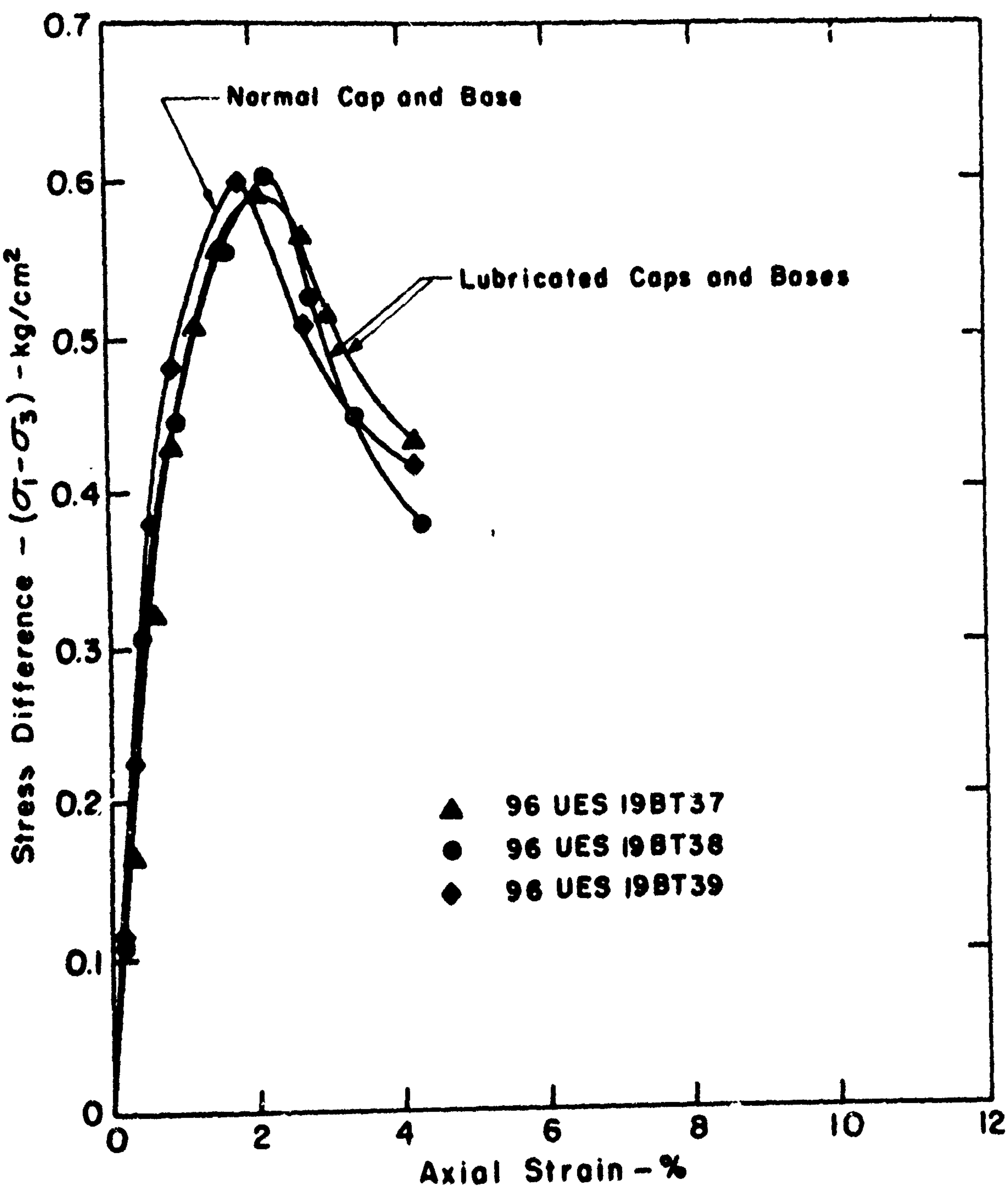


Fig. 5 STRESS-STRAIN CURVES FOR STRAIN-CONTROLLED TRIAXIAL TESTS ON VERTICAL SPECIMENS.

It may be noted that two of the stress-strain curves shown in Fig. 5 are for tests conducted using lubricated caps and bases to reduce the friction between the specimen ends and the end platens. The results of these tests were found to be essentially the same as for tests conducted using normal (unlubricated) caps and bases, however. As found previously from the results of studies on San Francisco Bay Mud, neither the strength nor the stress-strain behavior of saturated clays measured in unconsolidated-undrained (Q) tests appears to be significantly affected by cap and base restraint (Duncan, Seed, and Dunlop, 1966). The results of eight strain-controlled tests conducted to evaluate the effects of cap and base restraint, which are summarized in Table 3, show that there is no discernable difference between the results of tests conducted using lubricated or normal caps and bases. On the basis of these results, it was decided to conduct the remaining triaxial and plane strain tests on the Atchafalaya clay using normal caps and bases.

Plane Strain Tests on Vertical Specimens

The range of stress-strain curves for strain-controlled plane strain compression tests on vertical specimens is shown in Fig. 6. It may be noted that these stress-strain curves, like those for triaxial tests, reach a peak at very small values of strain, and that the shear resistance drops off with increasing strain beyond the peak. The results of these tests, summarized in Table 4, are in very close agreement with the results of triaxial compression tests on vertical specimens summarized in Table 3.

The average stress-strain curves for triaxial and plane strain tests, compared in Fig. 7, show that the peak strength values, the strains at failure, and the stress-strain characteristics are virtually the same for both types of tests, as found previously in the case of San Francisco Bay Mud (Duncan

Table 3. RESULTS OF STRAIN-CONTROLLED TRIAXIAL TESTS ON VERTICAL SPECIMENS

Sample	Water Content	Confining Pressure kg/cm^2	$(\sigma_1 - \sigma_3)_f$ kg/cm^2	ε_f Strain at Failure	Time to Failure Minutes	Caps and Bases
96UES18BT22	70.5%	1.5	0.549	2.2%	6	Normal
96UES18BT23	70.1%	1.5	0.530	2.1%	6	Normal
96UES18CT27	69.7%	1.5	0.592	3.0%	9.5	Lubricated
96UES18CT28	69.5%	1.5	0.607	3.4%	10	Lubricated
96UES18CT29	71.7%	1.5	0.608	3.3%	9.5	Normal
96UES19BT37	61.8%	1.5	0.589	2.1%	5.5	Lubricated
96UES19BT38	62.9%	1.5	0.601	2.1%	5.5	Lubricated
96UES19BT39	61.2%	1.5	0.597	1.8%	5	Normal
Average	67.2%	1.5	0.58	2.5%	7.1	---

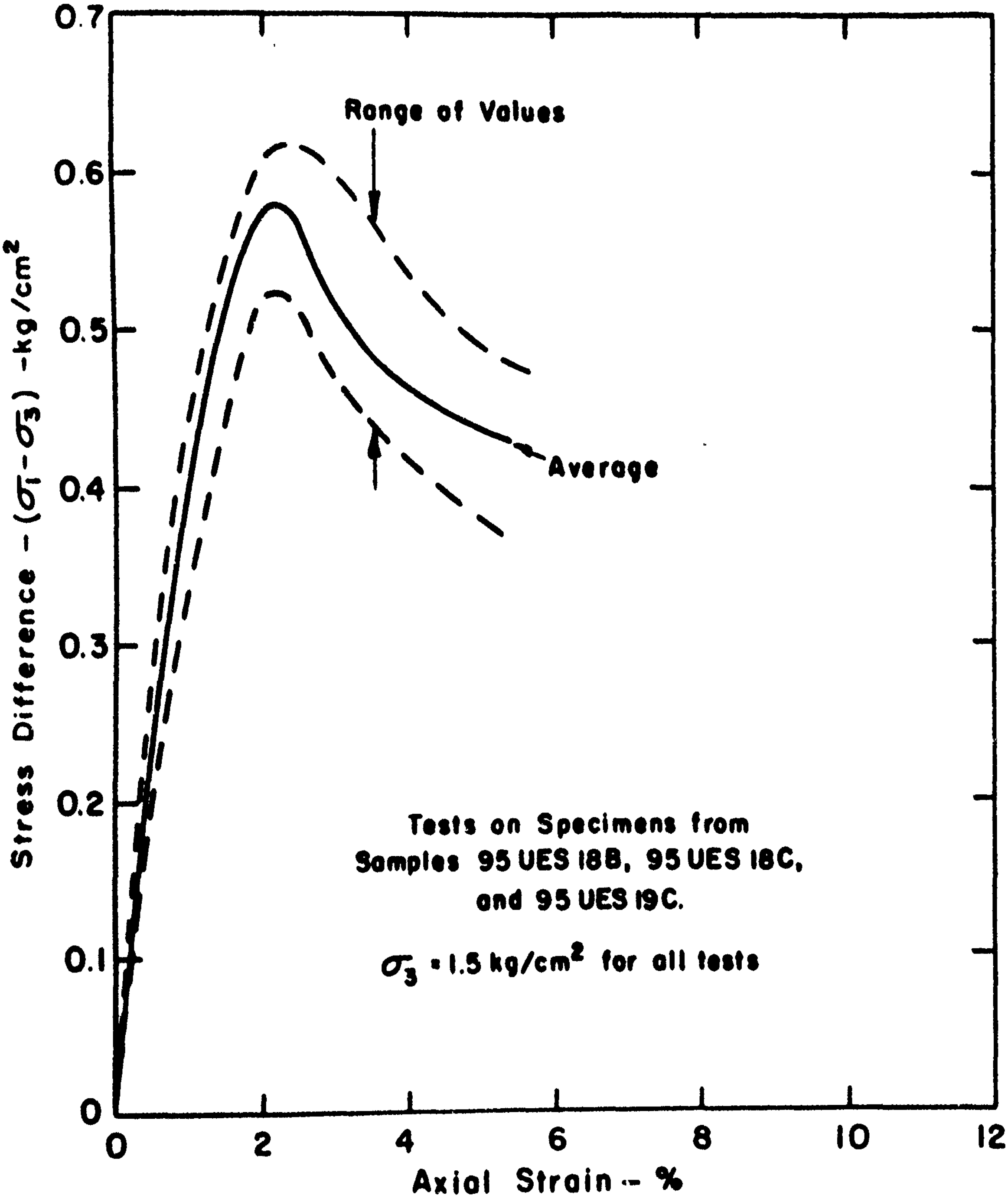


Fig. 6 RANGE OF STRESS-STRAIN CURVES FOR STRAIN-CONTROLLED PLANE STRAIN TESTS ON VERTICAL SPECIMENS.

Table 4. RESULTS OF PLANE STRAIN TESTS

Sample	Water Content	Confining Pressure kg/cm^2	$(\sigma_1 - \sigma_3)_f$ kg/cm^2	ε_f Strain at Failure	Time to Failure Minutes
95UES18BPS1	73.8%	1.5	0.591	2.2%	5
95UES18CPS3	68.0%	1.5	0.601	3.0%	7
95UES19CPS4	59.2%	1.5	0.593	1.8%	4
95UES19CPS5	60.6%	1.5	0.560	2.1%	5
96UES19CPS10	64.4%	1.5	0.560	1.8%	4
96UES19CPS12	59.1%	1.5	0.614	2.1%	5
96UES19CPS13	57.8%	1.5	0.608	2.1%	5
96UES19CPS14	60.5%	1.5	0.567	1.8%	4
96UES19CPS15	60.5	1.5	0.567	1.8%	4
Average	62.6%	1.5	0.58	2.1%	4.7

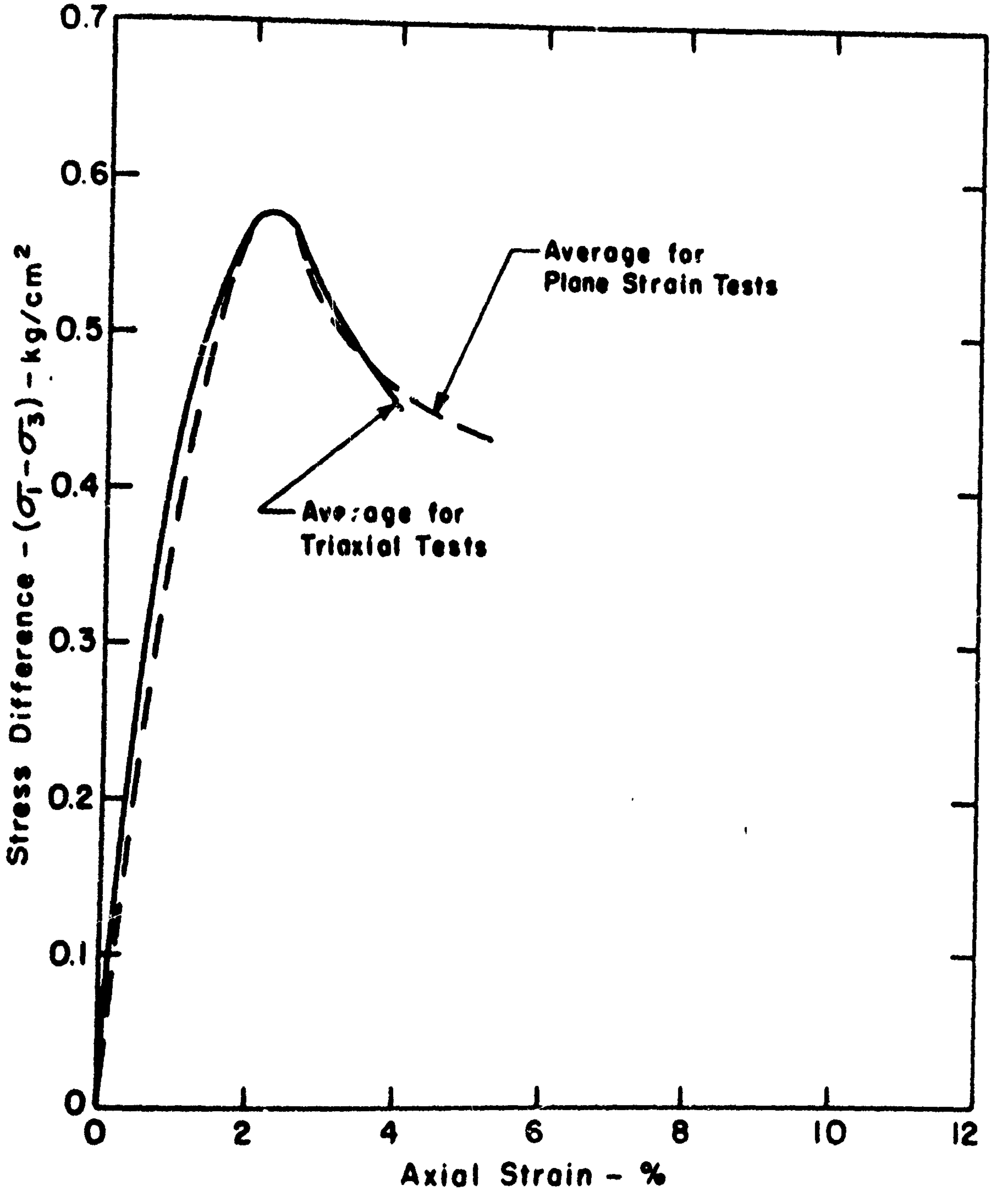


Fig. 7 COMPARISON OF AVERAGE STRESS-STRAIN CURVES FOR TRIAXIAL AND PLANE STRAIN TESTS ON VERTICAL SPECIMENS.

and Seed, 1965a). Thus it appears that it would be appropriate to use the results of triaxial tests for analyses of stability and movements of the Atchafalaya levees, even though the deformations in the field may correspond more closely to conditions of plane strain.

Triaxial Tests on Inclined and Horizontal Specimens

To study the variation of strength with failure plane orientation (or anisotropy) of the Atchafalaya clay, unconsolidated-undrained (Q) triaxial compression tests were conducted on specimens with various orientations. The orientations of these specimens are described in terms of the angle β, measured from the specimen axis to the in-situ horizontal plane. As described previously, a number of tests were conducted on vertical specimens, for which β was 90°. Tests were also conducted on specimens oriented with their axes at 60° and 30° to the horizontal plane, and on horizontal specimens (β = 0°). Stress-strain curves for tests on 60°, 30°, and horizontal specimens are shown in Figs. 8, 9, and 10, and the results of these tests are summarized in Table 5.

The variation of the measured values of peak stress difference, or compressive strength, with specimen orientation is shown in Fig. 11. It may be noted that the average compressive strength for horizontal specimens is about 7% higher than for vertical specimens. The minimum strength value, corresponding to a value of β between 30° and 60°, where the failure plane very nearly coincides with the in-situ horizontal plane, is about 7% lower than the strength of vertical specimens. The values of S_u/p corresponding to the results shown in Fig. 11 range from a maximum value of 0.25 for horizontal specimens to a minimum value of 0.22 for 30° and 60° specimens. Although the minimum strength corresponds to the condition where the failure

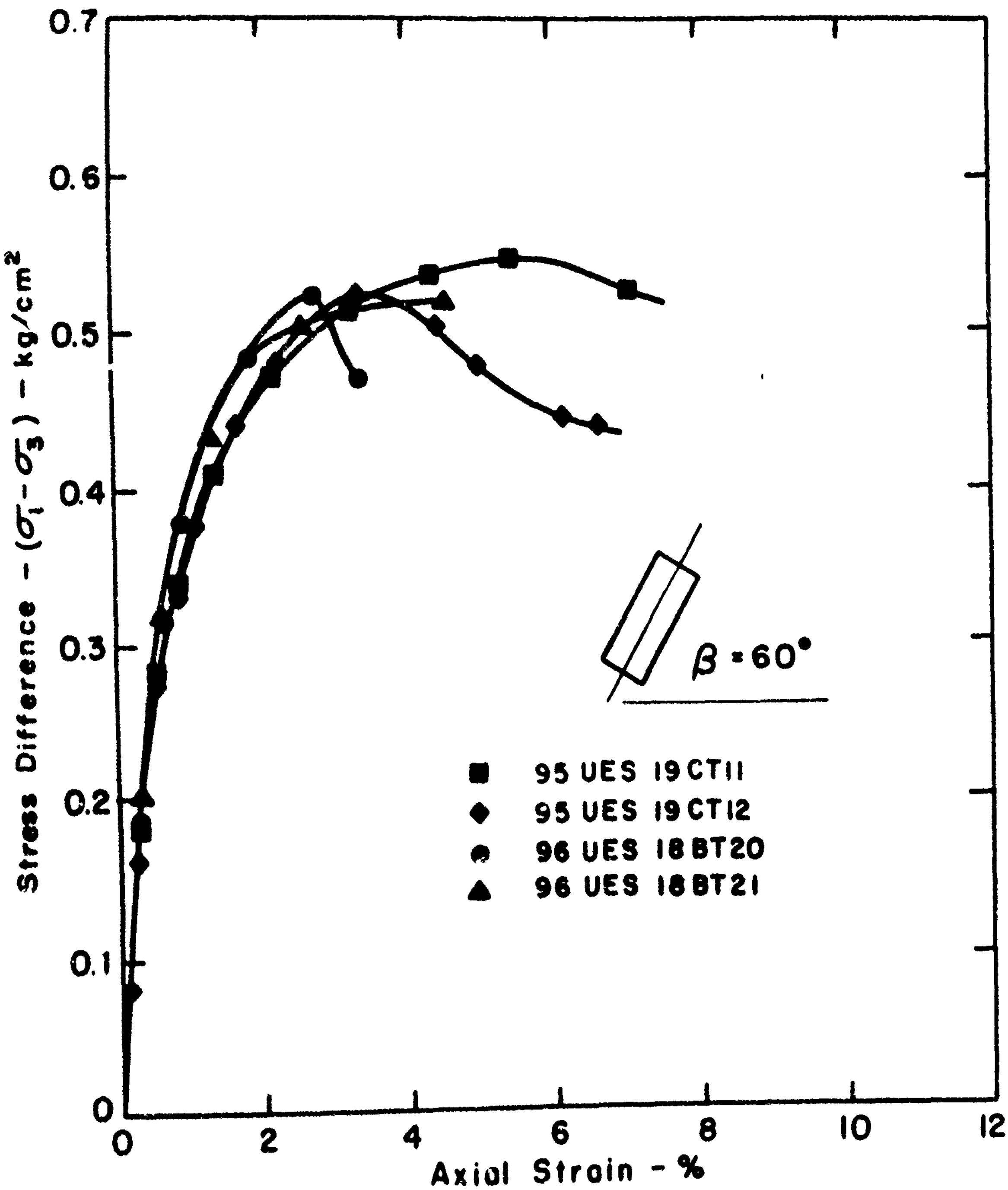


Fig. 8 STRESS-STRAIN CURVES FOR STRAIN-CONTROLLED TRIAXIAL TESTS ON 60° SPECIMENS.

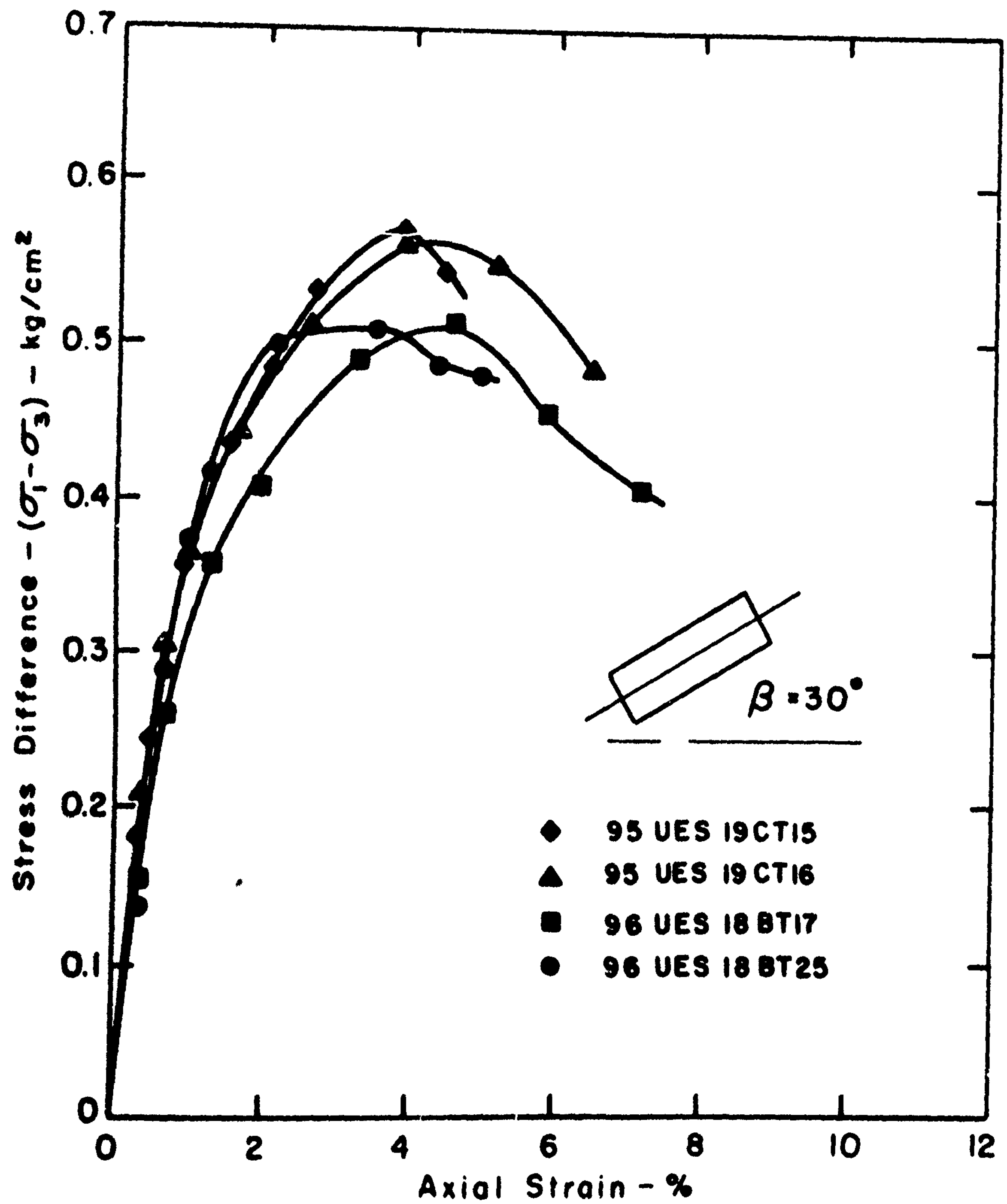


Fig. 9 STRESS-STRAIN CURVES FOR STRAIN-CONTROLLED TRIAXIAL TESTS ON 30° SPECIMENS.

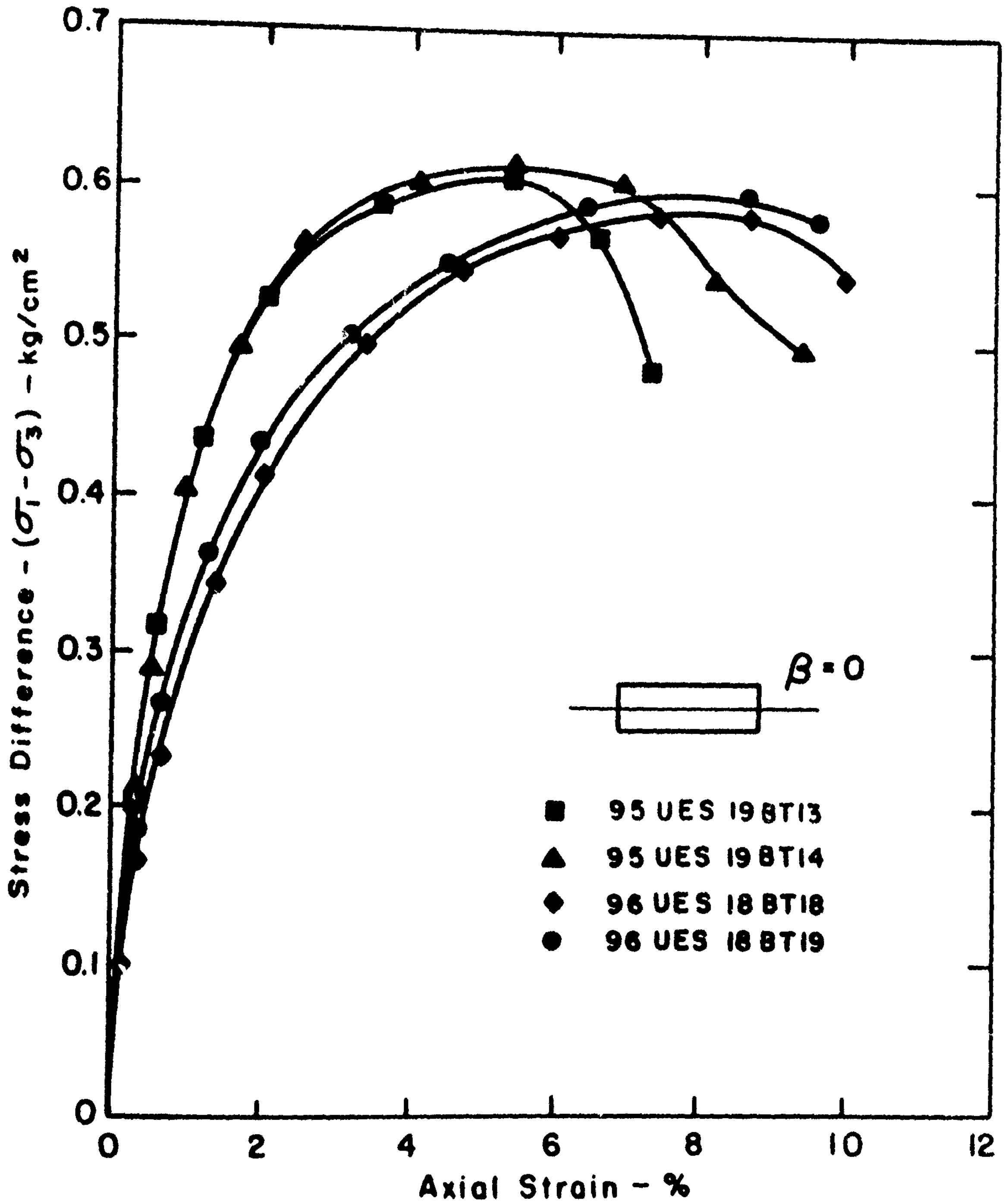


Fig. 10 STRESS-STRAIN CURVES FOR STRAIN-CONTROLLED TRIAXIAL TESTS ON HORIZONTAL SPECIMENS.

Table 5. RESULTS OF TESTS ON TRIAXIAL SPECIMENS

TRIMMED WITH VARIOUS ORIENTATIONS

Sample	Water Content	Confining Pressure kg/cm^2	$(\sigma_1 - \sigma_3)_f$ kg/cm^2	ε_f Strain at Failure	Time to Failure Minutes	β
95UES19CT11	61.4%	1.5	0.546	5.4%	16	60°
95UES19CT12	61.3%	1.5	0.521	3.6%	11	60°
96UES18BT20	75.4%	1.5	0.522	2.7%	7.5	60°
96UES18BT21	73.8%	1.5	0.533	3.9%	10	60°
95UES19CT15	60.4%	1.5	0.569	3.9%	11	30°
95UES19CT16	60.4%	1.5	0.539	3.9%	10	30°
96UES18BT17	66.0%	1.5	0.511	4.6%	11.5	30°
96UES18BT25	65.0%	1.5	0.508	3.4%	9	30°
95UES19BT13	69.2%	1.5	0.604	4.8%	13.5	0°
95UES19BT14	66.9%	1.5	0.615	5.3%	14.0	0°
96UES18BT18	67.7%	1.5	0.586	8.0%	20.0	0°
96UES18BT19	66.7%	1.5	0.595	8.6%	22.5	0°
96UES18BT22	70.5%	1.5	0.549	2.2%	6.0	90°
96UES18BT23	70.1%	1.5	0.530	2.1%	6.0	90°
96UES18CT27	69.7%	1.5	0.592	3.0%	9.5	90°
96UES18CT28	69.5%	1.5	0.607	3.4%	10	90°
96UES18CT29	71.7%	1.5	0.608	3.3%	9.5	90°
96UES19BT37	61.8%	1.5	0.589	2.1%	5.5	90°
96UES19BT38	62.9%	1.5	0.601	2.1%	5.5	90°
96UES19BT39	61.2%	1.5	0.597	1.8%	5.0	90°

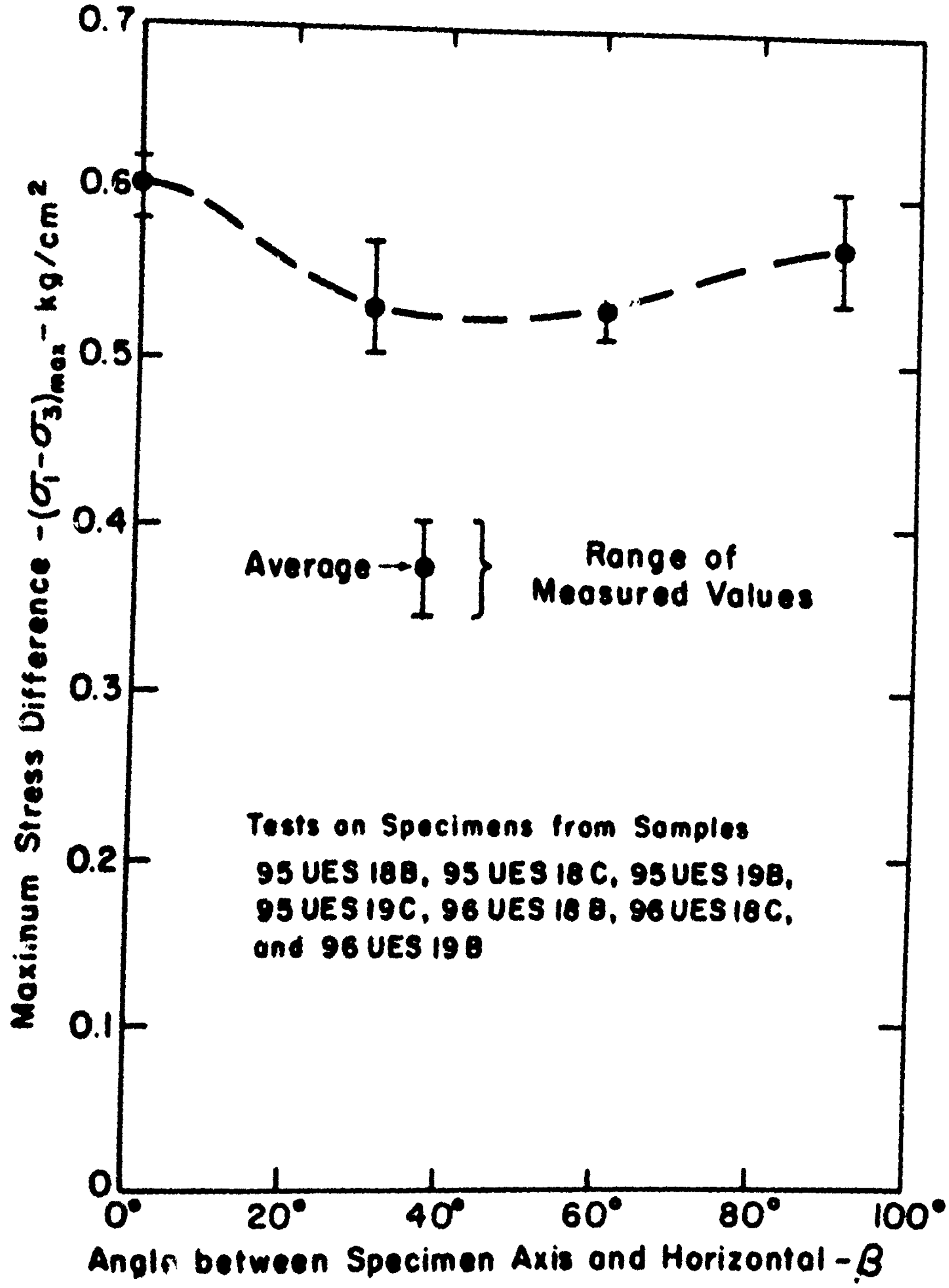


Fig. 11 VARIATION OF COMPRESSIVE STRENGTH WITH DIRECTION OF COMPRESSION.

plane is parallel to the layers of silt found in some specimens, the presence of the silt layers did not appear to have a large effect on the strength of the clay. It may be noted that the failure plane in the triaxial specimen shown in Fig. 1 is parallel to a light-colored layer of silt, but does not pass through the layer. Although failure planes were found to pass through silt layers in other specimens, these specimens were not significantly weaker or stronger than those in which the failure planes passed through the clay.

Average stress-strain curves for four different specimen orientations are shown in Fig. 12. To facilitate comparison of these curves, the ordinates have been normalized by dividing by the maximum value of stress difference so that the peak of each curve corresponds to 100%. It may be seen that the steepest stress-strain curve, the smallest value of strain at failure, and the most rapid reduction in shear resistance beyond the peak correspond to specimens oriented vertically, for which $\beta = 90°$. As the value of β decreases from 90° to zero, the stress-strain curves become progressively flatter, the values of strain at failure become larger, and the rate at which the shear resistance drops off beyond the peak decreases. On the basis of these results, it appears to be desirable to take into account variations in strength and stress-strain behavior of the foundation soils in analyses of the stability and movements of the Atchafalaya levees.

Triaxial Creep Tests

Four unconsolidated-undrained (Q) triaxial creep tests were performed on vertical specimens to study the time-dependent deformations of the Atchafalaya clay. Each of the tests was conducted using the same confining pressure (1.5 kg/cm^2) but different axial loads. The first specimen was

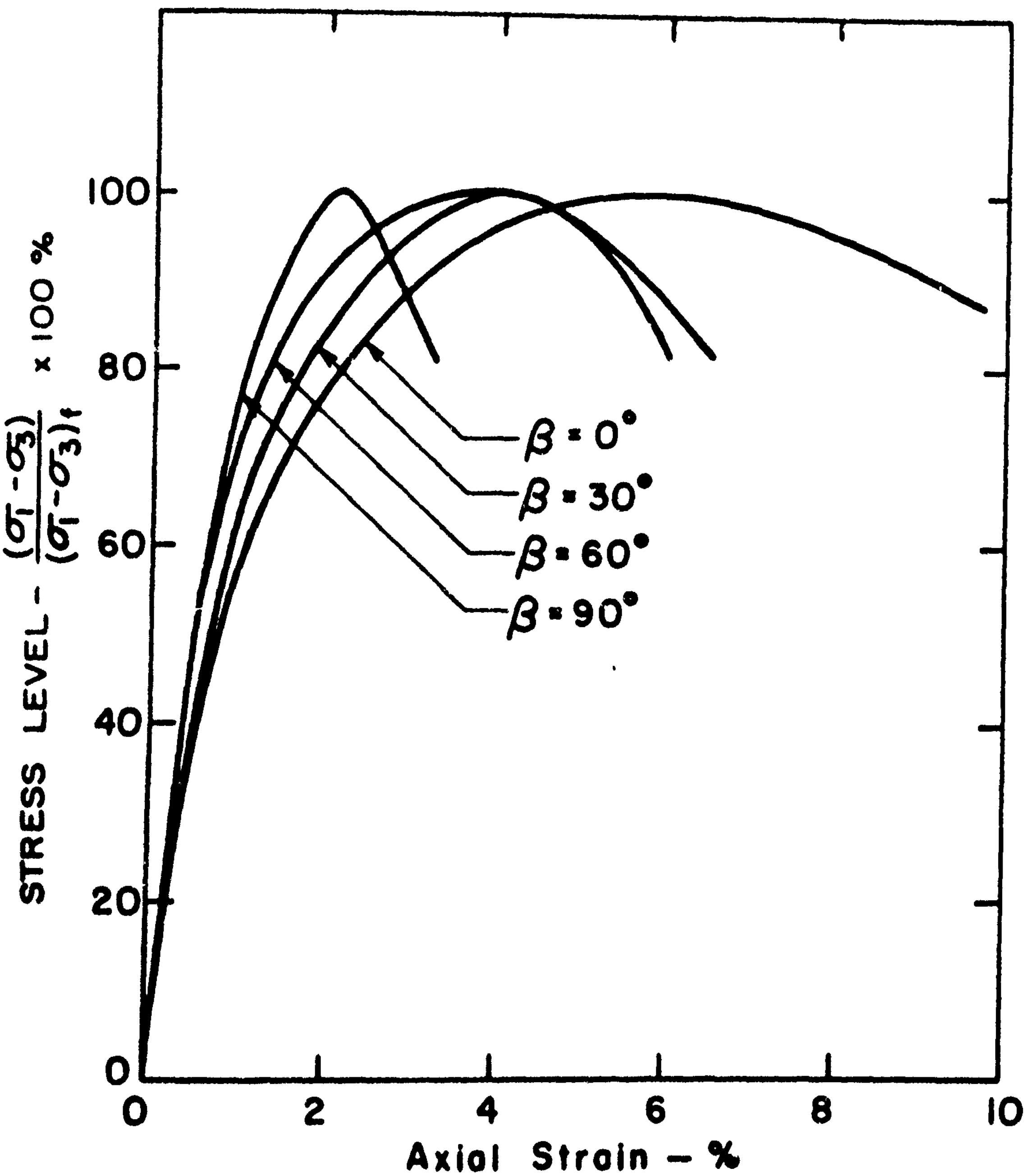


Fig. 12 COMPARISON OF NORMALIZED STRESS-STRAIN CURVES FOR SPECIMENS TRIMMED WITH FOUR DIFFERENT ORIENTATIONS.

loaded to a value of stress difference equal to 0.575 kg/cm^2, or 100% of the strength measured in short-term tests. Other specimens from the same section of sample were loaded to 0.525 kg/cm^2 (91% of the short term strength), 0.475 kg/cm^2 (83%), and 0.425 kg/cm^2 (74%). The loads were maintained constant throughout the tests, but as the specimens deformed and their cross-sectional areas increased, the values of stress difference decreased slightly, by 1% to 4%.

The first specimen, loaded initially to 100%, failed after 10 minutes. The other specimens deformed considerably, but did not fail. These results differ considerably from the results of a series of tests of a preliminary nature (Kaufman and Weaver, 1967), which showed that the undrained strength decreased linearly with the logarithm of load duration. Their results suggested that if the load was maintained for one year, the undrained strength would be only 20% as great as that measured in short-term tests. Additional studies will be required to determine if this difference in behavior results from variations of the properties of the clay with depth beneath the surface or from other factors. The variations of axial strain with time for the creep tests are shown in Fig. 13, where it may be noted that the deformations increase approximately linearly with the logarithm of time, the rate of deformation being largest for the most highly stressed specimen and smallest for the specimen having the lowest stress level.

The magnitude of these time-dependent deformations is quite appreciable compared to the deformations at comparable stress levels in short-term tests. The variations of strain shown in Fig. 13 are depicted in a different manner in Fig. 14, wherein curves showing variations of stress with strain for different periods of elapsed time are shown. Besides the stress-strain curves derived from creep test results, the average stress-strain curve

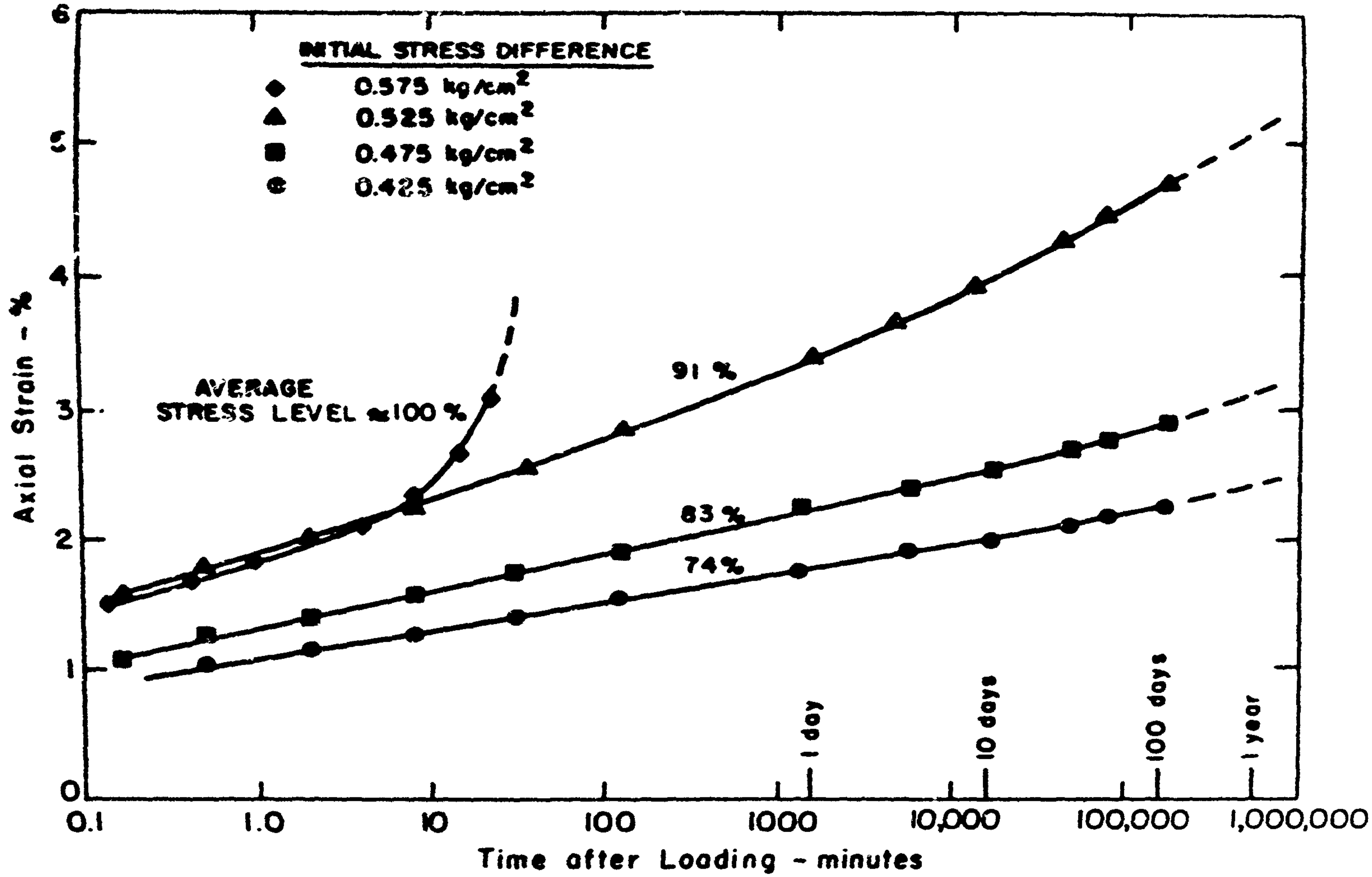


Fig. 13 VARIATIONS OF AXIAL STRAIN WITH TIME FOR SPECIMENS LOADED TO VARIOUS STRESS LEVELS.

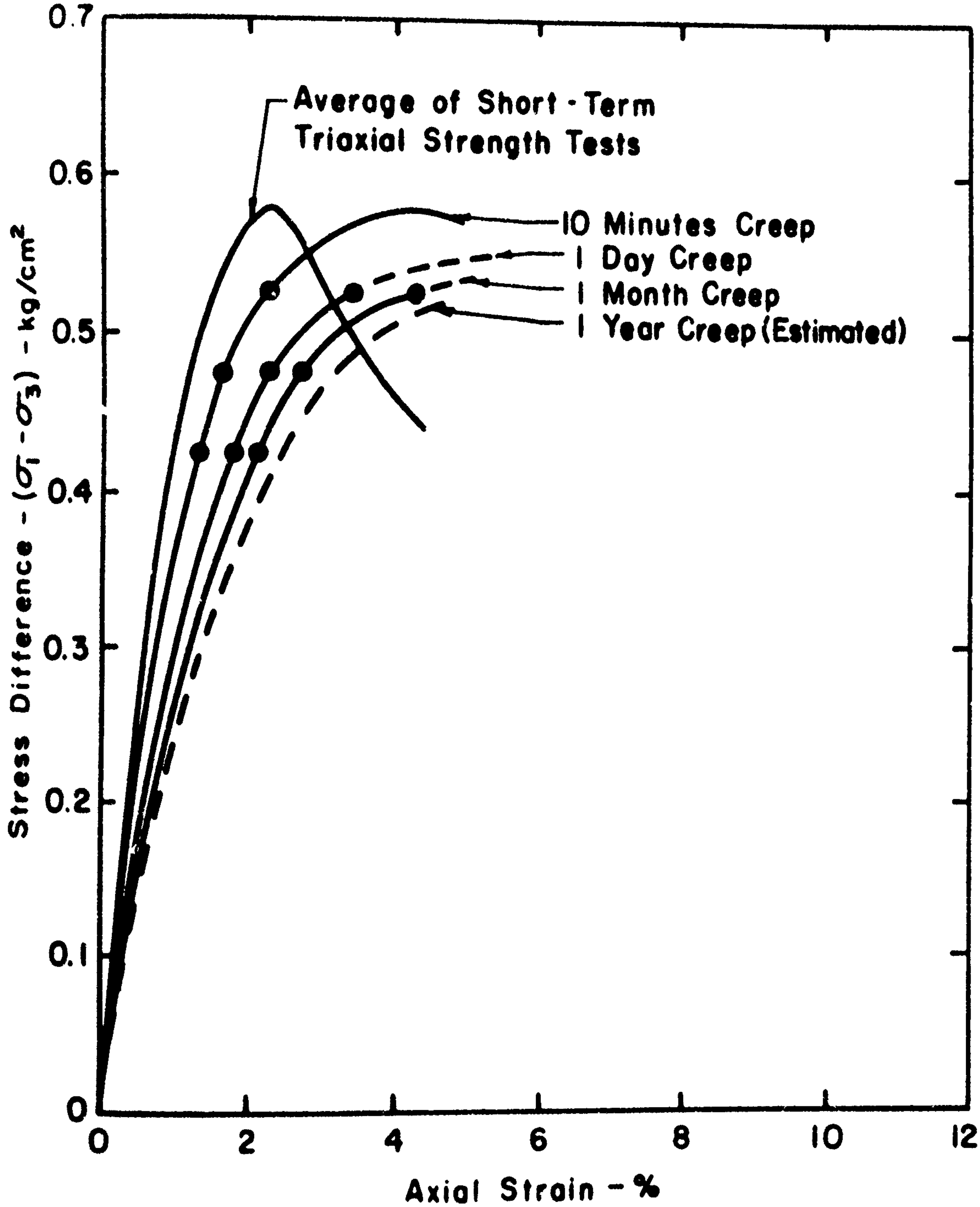


Fig. 14 STRESS-STRAIN CURVES FROM STRENGTH TESTS AND CREEP TESTS.

from the triaxial strength tests on vertical specimens is also shown in the figure. It may be noted that the values of strain from the strength tests are somewhat smaller at each stress level than those corresponding to creep for 10 minutes. There are two reasons for this difference: (1) The average duration of the triaxial strength tests was about 7 minutes, and (2) The loads were built up continually throughout the strength tests, whereas they were applied very rapidly at the beginning of the creep tests. Therefore, it would be expected that the values of strain from the creep test results would be somewhat larger than those from the strength tests at corresponding stress levels.

These results show that the duration of loading has a considerable effect on the stress-strain behavior of the Atchafalaya clay. The influence of duration of loading on soil modulus values should therefore be considered in determining properties for use in finite element analyses of stresses and deformations in these clays. One method of including the effects of load duration in such analyses would be to use creep curves of the type shown in Fig. 13 to estimate stress-strain curves for the duration of loading represented by the analysis, such as 1 year, 5 years, or 20 years. Using these stress-strain curves, nonlinear stress-strain parameters could be derived using the procedures developed by Duncan and Chang (1969), and used in finite element analyses to predict eventual movements.

Geonor Simple Shear Tests

Geonor simple shear tests were performed on the Atchafalaya clay using equipment manufactured by the Norwegian Geotechnical Institute, and testing procedures developed by the Norwegian Geotechnical Institute. Stress-strain curves for two Geonor simple shear tests are shown in Fig. 15. At the time

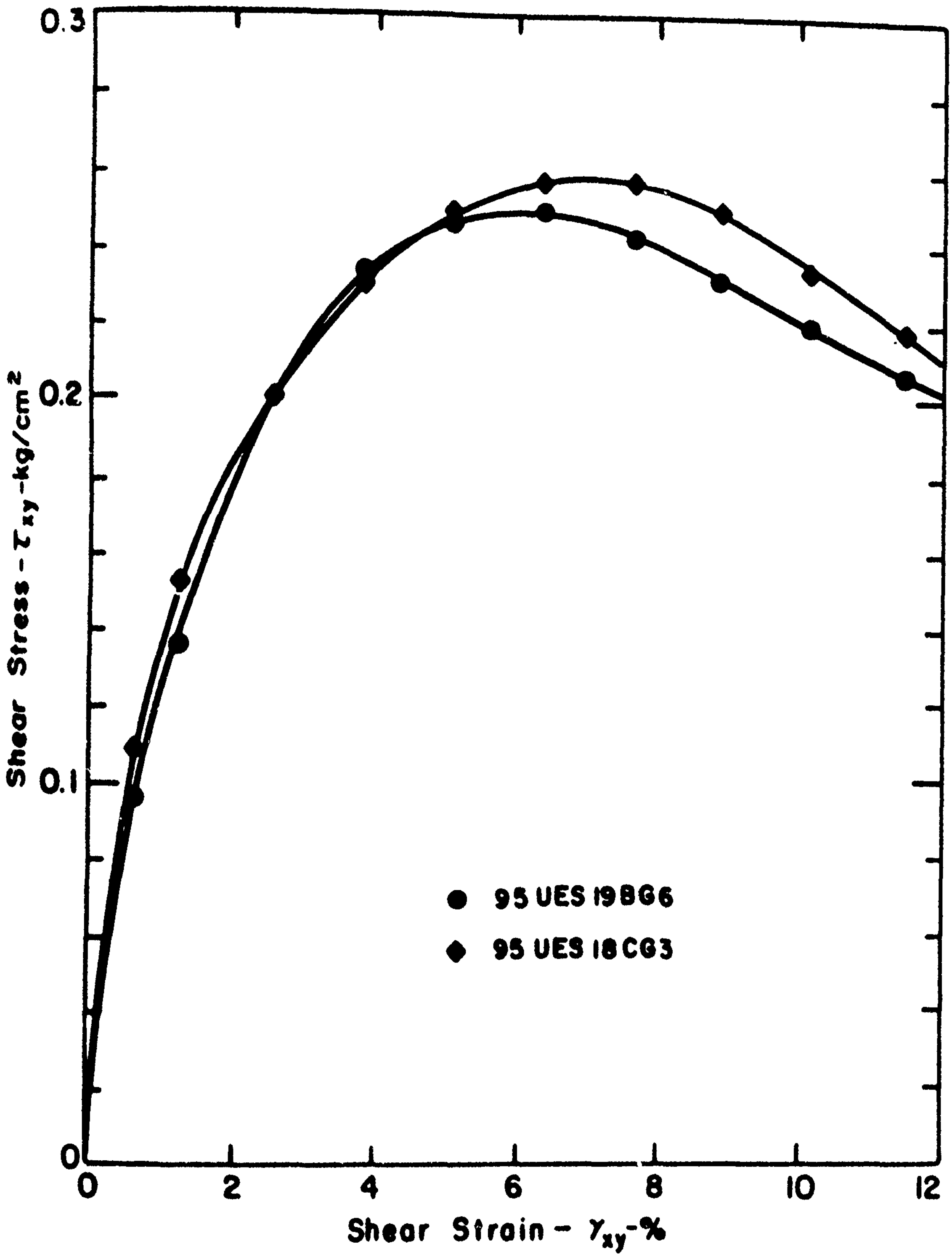


Fig. 15 STRESS-STRAIN CURVES FOR GEONOR SIMPLE SHEAR SPECIMENS TESTED AT 5% SHEAR STRAIN PER MINUTE.

these tests were conducted, the slowest rate of strain which could be achieved was 5% shear strain per minute, corresponding to test durations less than two minutes. To achieve test durations more nearly comparable with those employed in triaxial and plane strain strength tests, a new drive motor was installed on the Geonor simple shear apparatus so that the testing rate could be reduced to 1% shear strain per minute. Additional tests, conducted using this slower rate of strain, resulted in appreciably smaller values of τ_{xy} at failure, as shown in Fig. 16 and Table 6: The average value of τ_{xy} at failure measured in tests conducted at 5% shear strain per minute is 0.25 kg/cm^2, whereas that measured in tests conducted at 1% shear strain per minute is 0.22 kg/cm^2, a difference of about 12%. As discussed in later sections, the latter value is in good agreement with the results of Cambridge simple shear tests and triaxial tests conducted at comparable rates of strain.

It may be noted that the stress-strain curves shown in Fig. 16 indicate an appreciable drop in shear resistance beyond the peak. In fact, the shear resistance of some specimens was found to decrease to zero with increasing strain beyond the peak. This aspect of the stress-strain behavior was found to be due to the force component parallel to the failure plane which resulted from the vertical load applied to the test specimens. As shown in Fig. 17, a slip plane was found to develop in Geonor simple shear specimens about the time the peak value of τ_{xy} was reached. This slip plane was not horizontal, but dipped in the direction of applied shear stress (to the right in Fig. 17), passing from the cap at one side of the specimen to the base at the other side*. As shown in Fig. 17, the component of the applied

*Duncan and Dunlop (1969) have shown that the failure plane in simple shear tests would be expected to be inclined more steeply, at 35° or 40° to the horizontal. ... of the observed slip planes are

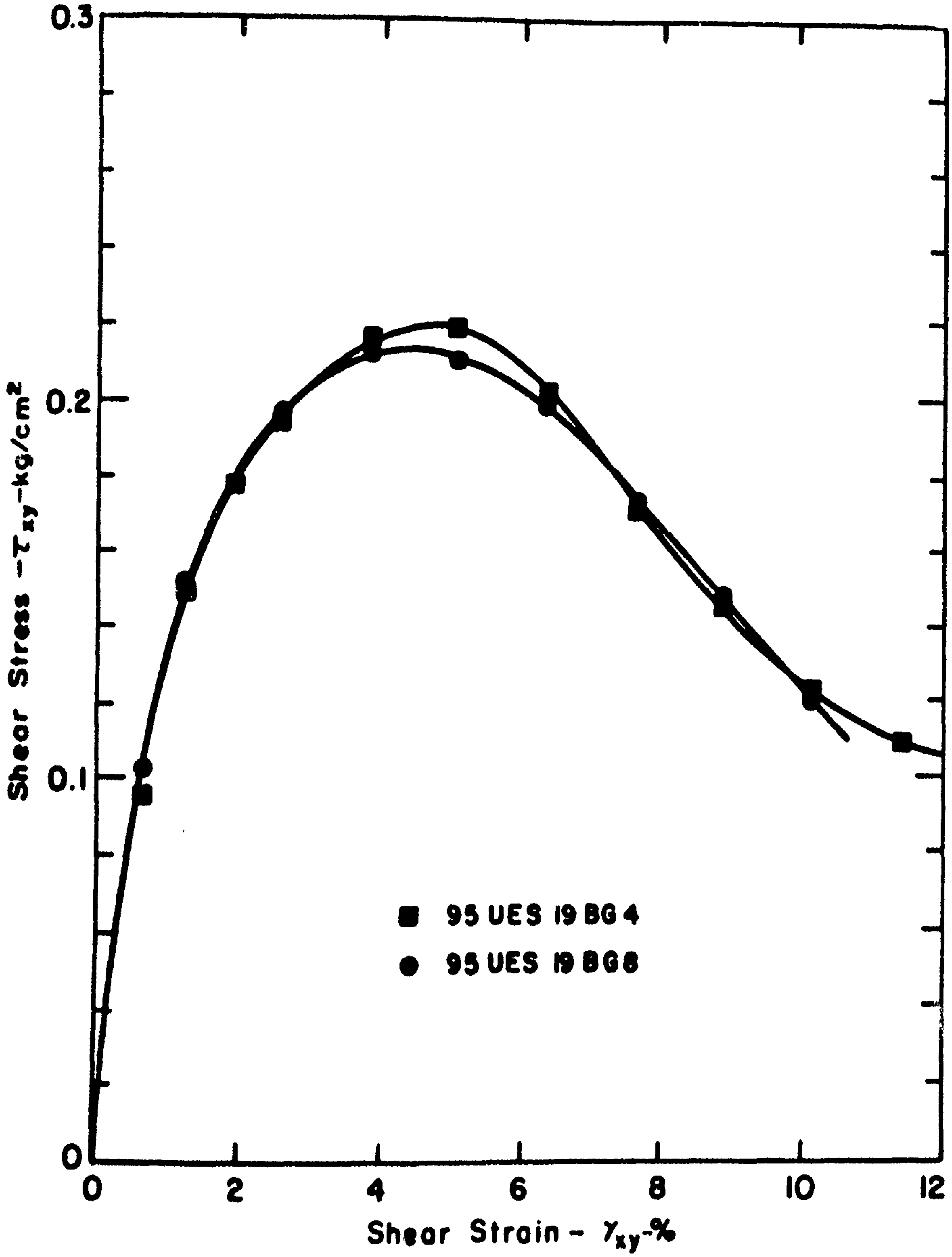


Fig. 16 STRESS-STRAIN CURVES FOR GEONOR SIMPLE SHEAR SPECIMENS TESTED AT 1% SHEAR STRAIN PER MINUTE.

Table 6. RESULTS OF GEONOR SIMPLE SHEAR TESTS

Sample	Water Content	Normal Stress kg/cm^2	$(\tau_{xy})_{max}$ kg/cm^2	(γ_{xy}) at fialure	Time to Failure Minutes
95UES18CG3	62.7%	3.0	0.258	7%	1.5
95UES19BG6	62.4%	3.0	0.250	6%	1.5
95UES19BG4	69.7%	3.0	0.219	5%	6
95UES19BG8	67.2%	3.0	0.212	4%	4

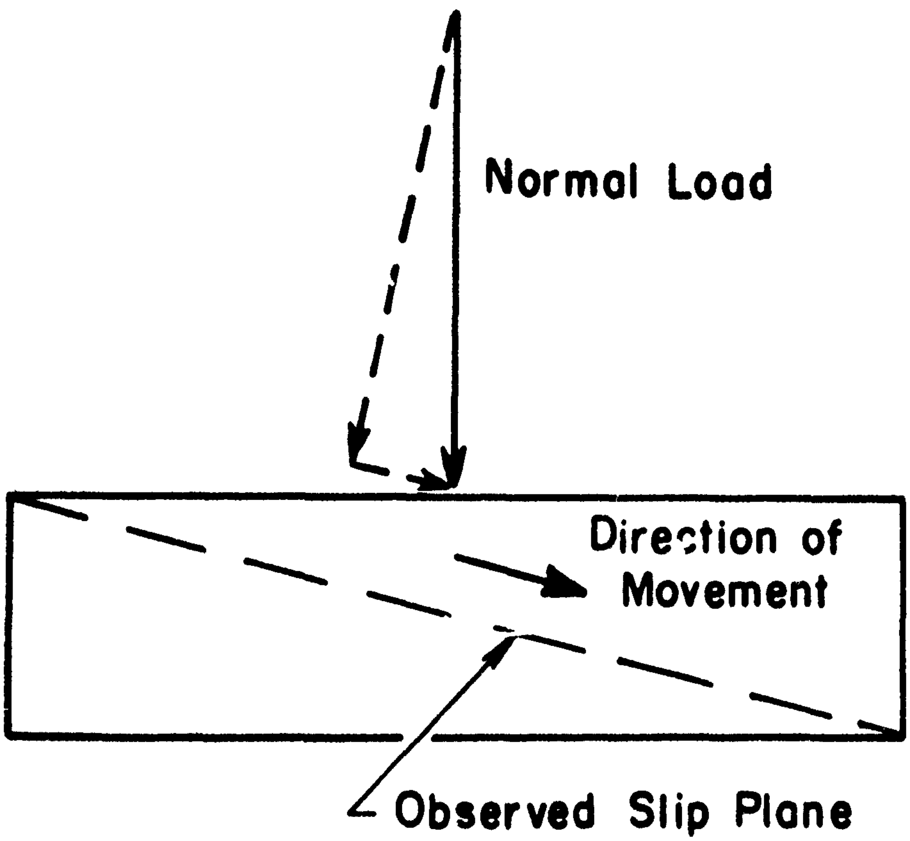


Fig. 17 FAILURE MECHANISM IN GEONOR SIMPLE SHEAR TESTS.

normal load parallel to this slip plane acts in the direction of movement, and reduces the magnitude of the horizontal force required to produce movement. The effects of the normal loads and initial stress conditions on the peak shear resistance in simple shear tests is discussed more fully subsequently, in connection with the comparison of simple shear and triaxial test results.

In some cases, Geonor simple shear specimens were found to undergo premature failure by slipping along the contact between the specimen and the cap or base. The stress-strain curve for one such test is shown in Fig. 18. It may be seen that the maximum value of τ_{xy} for this test is about 0.18 kg/cm^2, or roughly 20% less than that for specimens tested at the same rate of strain which failed through the interior. The problem of failure by sliding at the cap or base was found to result from the fact that the sandpaper with which the cap and base were covered weakened when in contact with water, and use of waterproof emery cloth, bonded to the cap and base with waterproof glue, was found to eliminate this problem. Because slip between the specimen and the cap or base has such an important influence on the shear characteristics, it is necessary to examine each specimen carefully to insure that failure occurred through the interior and not by sliding at top or bottom.

<u>Cambridge Simple Shear Tests</u>

Stress-strain curves for three Cambridge simple shear tests are shown in Fig. 19, and the results of these tests are summarized in Table 7. It may be noted that the stress-strain curves for these three tests differ quite appreciably, and that the highest and lowest measured values of τ_{xy} at failure differ by almost 15%. This variation in test results is

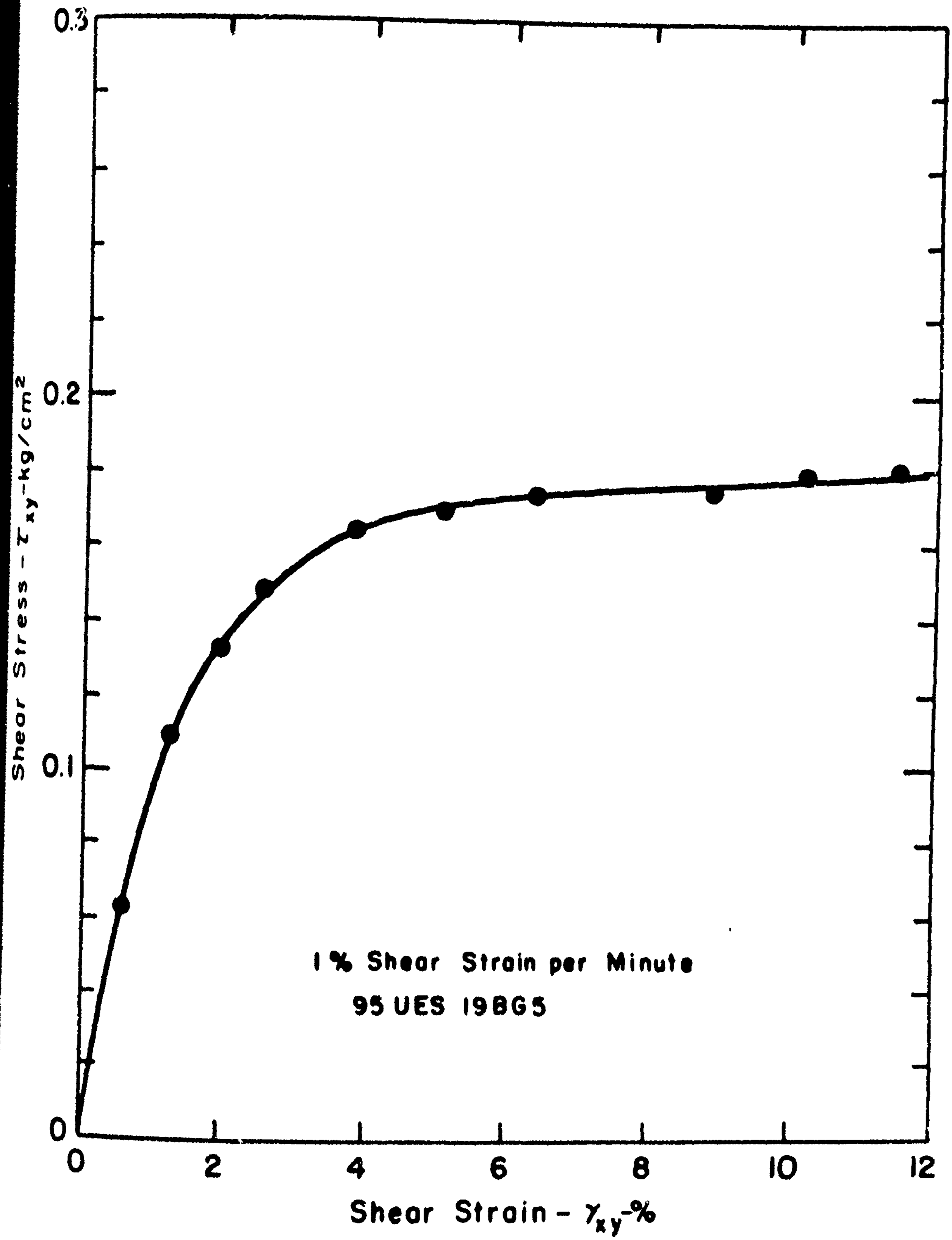


Fig. 18 STRESS-STRAIN CURVE FOR GEONOR SIMPLE SHEAR SPECIMEN WHICH FAILED BY SLIDING ON CAP.

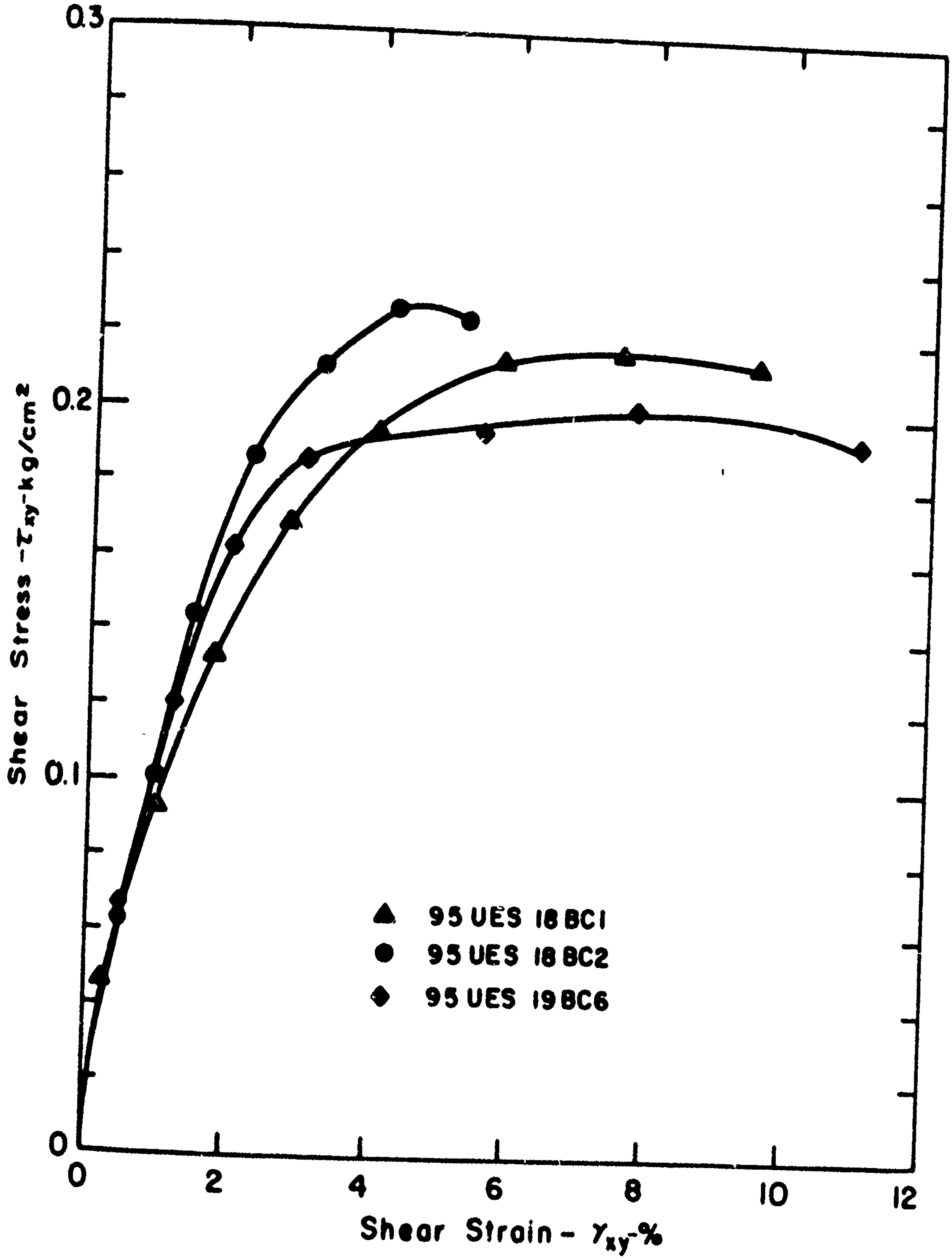


Fig. 19 STRESS-STRAIN CURVES FOR CAMBRIDGE SIMPLE SHEAR SPECIMENS TESTED AT 1% SHEAR STRAIN PER MINUTE.

Table 7. RESULTS OF CAMBRIDGE SIMPLE SHEAR TESTS

Sample	Water Content	Normal Stress kg/cm^2	$(\tau_{xy})max$ kg/cm^2	(γ_{xy}) at failure	Time to Failure Minutes
95UES18BC1	62.9%	1.5	0.217	6.5%	12
95UES18BC2	62.8%	1.5	0.227	4.5%	10
95UES18BC6	59.7%	1.5	0.200	8%	14
Average	61.8%	1.5	0.215	6%	12

considerably greater than for Geonor simple shear or triaxial or plane strain tests, and is believed to result from friction in the Cambridge shear apparatus The frictional force, measured by testing "specimens" of water, amounted to as much as 20% of the total measured shear load during a test, and, as shown in Fig. 20, this frictional resistance fluctuated considerably as the shearing deformation increased. The friction measured during the first tests conducted to calibrate the Cambridge simple shear machine was even higher. The friction was reduced to the level shown in Fig. 20 by disassembling and realligning the apparatus. It is thus evident that the magnitude of the frictional resistance must be reevaluated each time the apparatus is realigned. Although the calibrations were performed carefully and appropriate corrections were applied to the measured values of shear force as shown in Fig. 20, the fact that the required corrections were quite large and fluctuated with shear deformation is probably responsible for some of the variations in test results shown in Fig. 19.

The average stress-strain curves measured in Geonor simple shear and Cambridge simple shear tests are shown in Fig. 21. The two curves are quite similar, and in view of the fact that there is some uncertainty regarding the corrections appropriate for the Cambridge simple shear tests, it is considered that the results of the two types of tests are the same for all practical purposes.

COMPARISON OF COMPRESSION TEST AND SIMPLE SHEAR TEST RESULTS

Shear Strength

The average value of shear strength of the Atchafalaya clay measured in triaxial and plane strain compression tests (0.28 kg/cm^2), is considerably

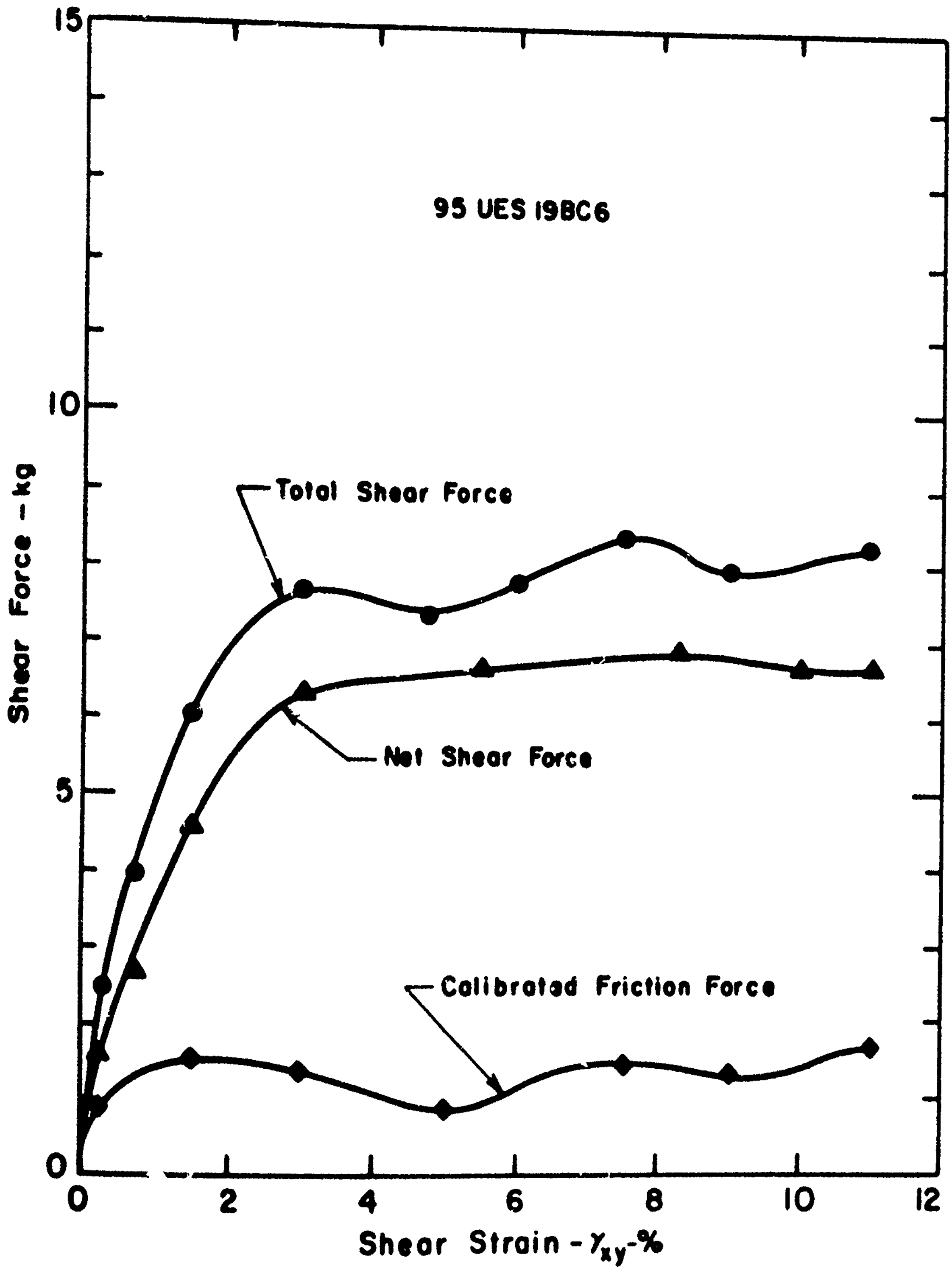


Fig. 20 VARIATIONS OF SHEAR RESISTANCE AND FRICTIONAL RESISTANCE WITH SHEAR STRAIN.

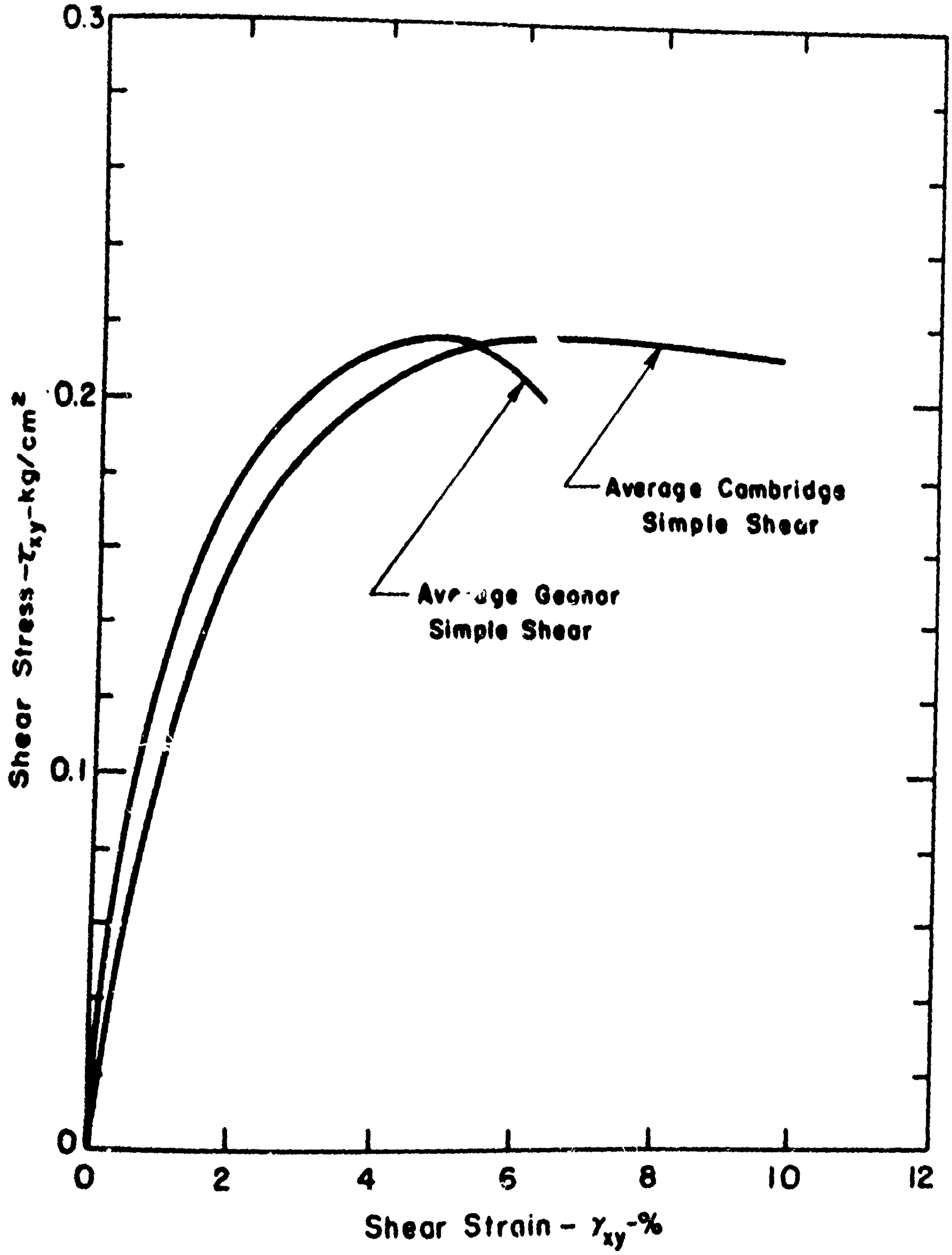


Fig. 21 COMPARISON OF AVERAGE STRESS-STRAIN CURVES FOR GEONOR AND CAMBRIDGE SIMPLE SHEAR SPECIMENS TESTED AT 1% SHEAR STRAIN PER MINUTE.

greater than the maximum value of shear resistance (τ_{xy}) measured in simple shear tests (0.22 kg/cm^2). A recent study of the behavior of soils in simple shear tests (Dunlop, Duncan and Seed, 1968; Duncan and Dunlop, 1969) has shown that the shear resistance of simple shear specimens (the maximum value of τ_{xy}) would be expected to be smaller than the shear strength of the soil tested if, as is the usual case, shear stresses are induced in the test specimens by application of the normal load. For purposes of interpreting the results of simple shear tests, it has been shown to be sufficiently accurate to assume that the stress conditions within the test specimen are completely uniform, in which case the simple shear stress conditions correspond precisely with those of pure shear. This assumption affords a simple means of relating the results of simple shear tests and plane strain or triaxial compression tests.

The initial values of effective normal stress on horizontal and vertical planes within simple shear specimens (σ_{yo}' and σ_{xo}') are in general not equal. In the case of consolidated-undrained (R) simple shear tests as described by Dunlop, Duncan, and Seed (1968), in which the specimens are consolidated one-dimensionally prior to shear, the initial stresses correspond to at-rest pressure conditions, with $\sigma_{xo}' = K_o\sigma_{yo}'$, in which K_o is the coefficient of at-rest earth pressure. In the case of unconsolidated-undrained (Q) simple shear tests as described in this report, the values of σ_{yo}' and σ_{xo}' depend on the values of effective stress which acted on the specimens in-situ and the magnitude of the change in effective stress which results from sampling, transportation, and trimming of the specimens. Specimens obtained from areas of horizontal ground were, by definition, subjected to in-situ stresses of p_o' in the vertical direction and K_op_o' in the horizontal direction, in which p_o' is the effective overburden pressure. If the clay

behaved elastically during release and reapplication of the stresses, and the necessary sampling, transportation and trimming operations were accomplished without disturbance, the initial effective stresses prior to the test would be the same as in the ground, with $\sigma_{yo}' = p_o'$, and $\sigma_{xo}' = K_o p_o'$. More typically, however, clay does not behave elastically during release and reapplication of stresses, and some amount of disturbance during the necessary handling is inevitable. As a result, the initial values of effective stress will be somewhat smaller than the in-situ values. If it is assumed that the values of σ_{yo}' and σ_{xo}' are reduced in equal proportion, these values may be expressed as

$$\sigma_{yo}' = \alpha\, p_o' \qquad (1)$$

$$\sigma_{xo}' = \alpha\, K_o p_o' \qquad (2)$$

in which α is a number smaller than unity which represents the ratio of the initial values of effective stress in the laboratory specimen to the in-situ values.

The stress conditions within simple shear test specimens initially, before application of the shear stress to the horizontal plane, are represented by the Mohr's circle labelled "before change" in Fig. 22. At this stage, the horizontal and vertical planes are the principal planes, and the stresses σ_{yo}' and σ_{xo}' are the major and minor principal stresses. When the shear stress (τ_{xy}) is applied, both the magnitudes and the orientations of the principal stresses are changed, even though the normal stresses on horizontal and vertical planes remain the same. Assuming that simple shear may be adequately represented by pure shear, the stress condition after application of the horizontal shear stress is shown by the larger Mohr's circle of stress in Fig. 22.

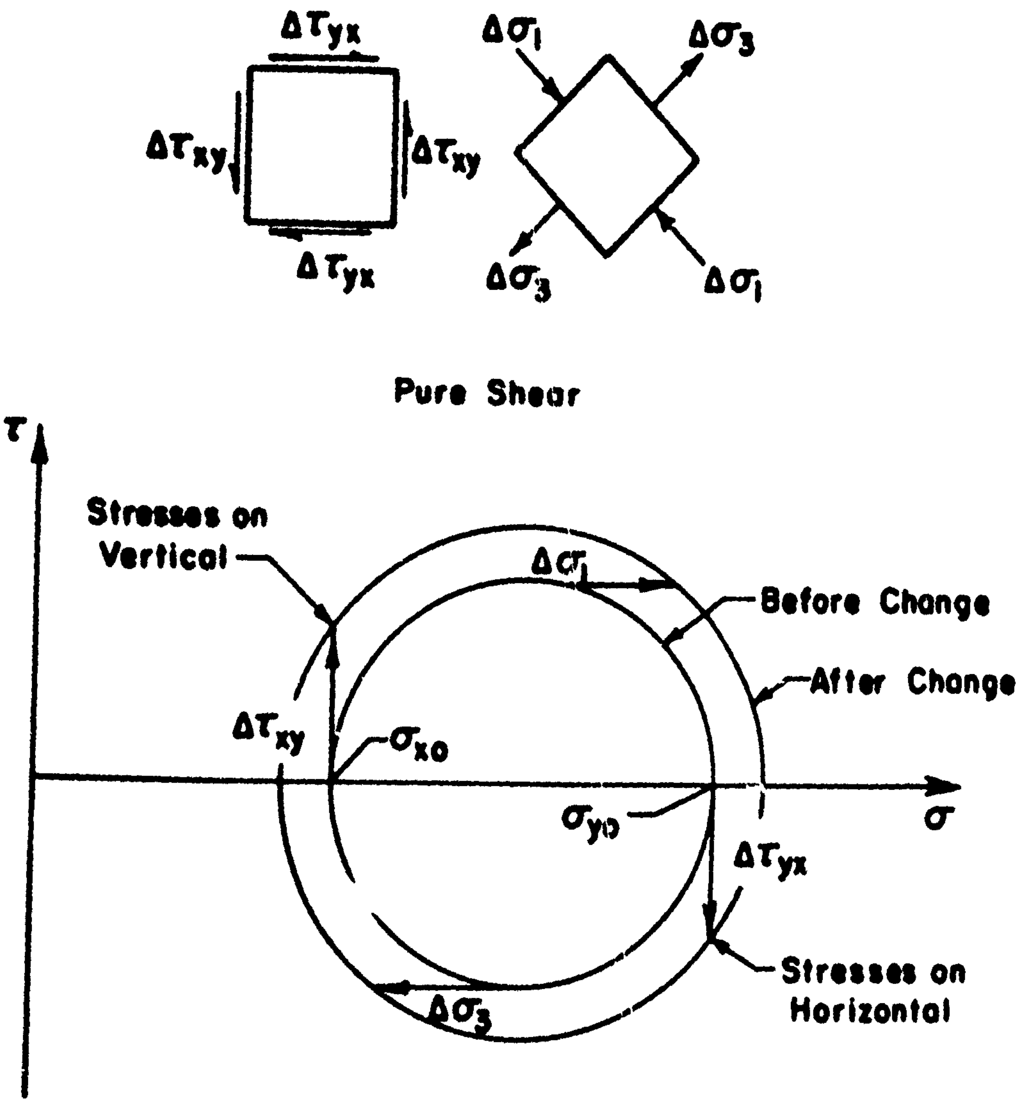


Fig. 22 PURE SHEAR STRESS CONDITIONS.

To make appropriate comparisons between the shearing behavior as determined in triaxial or plane strain tests and that determined in simple shear tests, the results of both types of tests should be compared on a consistent basis. The value of maximum shear stress at failure, $(\tau_{max})_f$, which is frequently employed as a measure of the shear strength of saturated clays, affords a suitable basis for comparing the results of the compression tests and simple shear tests. For triaxial or plane strain tests, in which the values of the principal stresses at failure are known, the value of τ_{max} may be calculated as follows

$$(\tau_{max})_f = \frac{1}{2}(\sigma_1 - \sigma_3)_f \qquad (3)$$

in which τ_{max} is the maximum shear stress, σ_1 and σ_3 are the major and minor principal stresses, and the subscript f denotes failure conditions. If, as described previously, it is assumed that the stresses within simple shear specimens correspond to pure shear stress conditions, the value of τ_{max} at failure may be expressed in terms of the initial stresses and the applied shear stresses as

$$(\tau_{max})_f = \sqrt{\left(\frac{\sigma_{yo}' - \sigma_{xo}'}{2}\right)^2 + (\tau_{xy})_f^{\,2}} \qquad (4)$$

in which σ_{yo}' and σ_{xo}' are the initial values of effective vertical and horizontal normal stress, and $(\tau_{xy})_f$ is the value of applied shear stress at failure. For use in comparing the results of unconsolidated-undrained (Q) triaxial and simple shear tests of the type described in this report, it is convenient to express σ_{yo}' and σ_{xo}' in terms of the in-situ stresses as shown by equations (1) and (2), in which case equation (4) may be written as

$$(\tau_{max})_f = \sqrt{\left[\frac{\alpha\, p_o'(1 - K_o)}{2}\right]^2 + (\tau_{xy})_f^2} \qquad (5)$$

in which, as explained previously, p_o' is the effective overburden pressure in-situ, K_o is the coefficient of earth pressure at rest, and α is a pure number, whose value is less than or equal to unity, which expresses the amount of reduction in effective stress resulting from the removal and reapplication of stresses and from disturbance.

Using this equation, values of α and K_o have been calculated for which the values of τ_{max} determined from triaxial tests and simple shear tests are the same. For purposes of comparison, the value of τ_{max} for triaxial tests was taken as 0.27 kg/cm^2 which, as shown in Fig. 11, corresponds to the average shear strength of specimens trimmed with their axes at about 35° to the horizontal, the stress orientation which Duncan and Dunlop (1969) found for simple shear specimens at failure. Using the average value of $(\tau_{xy})_f$ from both Geonor and Cambridge simple shear tests (0.22 kg/cm^2), and the estimated value of p_o' at a depth of 70 ft (1.2 kg/cm^2), values of α and K_o were calculated for which the values of τ_{max} calculated from equation (5) agree with the values from triaxial and plane strain compression tests; these values are listed in Table 8. Using the approximation that $K_o \approx (1 - \sin \phi')$ together with the value of ϕ' determined by Kaufman and Weaver (1967), results in an estimated value of K_o equal to 0.64 for the Atchafalaya clay. It may be noted that this value of K_o corresponds to a value of α between 0.7 and 0.8, and would indicate that the effective stress in the simple shear test specimens at the time they were tested was reduced about 20% to 30% below the in-situ value as a result of removal and reapplication of stress and of disturbance. A reduction of effective

Table 8. COMBINATIONS OF VALUES OF K_o AND α FOR WHICH THE SHEAR STRENGTHS FROM TRIAXIAL AND SIMPLE SHEAR TESTS ARE THE SAME

K_o	α
0.74	1.0
0.71	0.9
0.67	0.8
0.63	0.7
0.57	0.6

stress of 20% to 30% corresponds to a strength reduction of only 5% to 7%. Thus, the specimens tested would be considered to be of good quality, with undrained strengths representative of those in-situ.

On the basis of these considerations it may be concluded that the shear resistance of the Atchafalaya clay measured in simple shear tests is in reasonable agreement with the shear strength measured in triaxial and plane strain compression tests.

Stress-Strain Behavior

To examine the correspondence between the stress-strain behavior of the Atchafalaya clay determined from triaxial test results and that determined from simple shear test results, the procedure for nonlinear stress analysis developed by Duncan and Chang (1969) has been employed to predict the shape of the pure shear stress-strain curve using the results of triaxial tests. The procedure for stress-strain analysis developed by Duncan and Chang (which has been described in some detail in recent reports to the Waterways Experiment Station by Clough and Duncan, 1969 and Kulhawy, Duncan, and Seed, 1969) approximates nonlinear stress-strain behavior with a series of linear segments. For each segment, the increments of stress and the corresponding increment of strain are assumed to be related by the generalized form of Hookes' law,

$$\Delta\varepsilon_x = \frac{1}{E_t}\left[\Delta\sigma_x - \nu(\Delta\sigma_y + \Delta\sigma_z)\right] \tag{6a}$$

$$\Delta\varepsilon_y = \frac{1}{E_t}\left[\Delta\sigma_y - \nu(\Delta\sigma_z + \Delta\sigma_x)\right] \tag{6b}$$

$$\Delta\varepsilon_z = \frac{1}{E_t}\left[\Delta\sigma_z - \nu(\Delta\sigma_y + \Delta\sigma_x)\right] \tag{6c}$$

$$\Delta\gamma_{xy} = \frac{2(1 + \nu)}{E_t} \Delta\tau_{xy} \tag{6d}$$

$$\Delta\gamma_{yz} = \frac{2(1 + \nu)}{E_t} \Delta\tau_{yz} \tag{6e}$$

$$\Delta\gamma_{zx} = \frac{2(1 + \nu)}{E_t} \Delta\tau_{zx} \tag{6f}$$

in which x, y, and z are orthogonal coordinate axes, $\Delta\varepsilon$ is an incremental normal strain, $\Delta\gamma$ is an incremental shear strain, $\Delta\sigma$ is an incremental normal stress, $\Delta\tau$ is an incremental shear stress, E_t is the tangent modulus value corresponding to the average stress conditions during the increment, and ν is Poisson's ratio.

The value of tangent modulus is related to the stress conditions during the increment by the equation

$$E_t = \left[1 - \frac{R_f(1 - \sin\phi)(\sigma_1 - \sigma_3)}{2\, c \cos\phi + 2\, \sigma_3 \sin\phi}\right]^2 K p_a \left(\frac{\sigma_3}{p_a}\right)^n \tag{7}$$

in which K is the modulus number, n is an exponent expressing the rate of change of modulus with change in confining pressure, c is the cohesion intercept, ϕ is the angle of internal friction, and R_f is the failure ratio -- the ratio of the stress difference at failure to the stress difference representing the asymptote to the hyperbola which approximates the actual stress-strain curve. p_a is atmospheric pressure expressed in the same units as E_t and σ_3. The parameters c and ϕ are the Mohr-Coulomb shear strength parameters and may be determined readily by constructing an envelope to the Mohr's circles of stresses at failure in a series of tests conducted at various confining pressures. For the Atchafalaya clay, confining pressure has no influence on the strength in unconsolidated-undrained tests; the value

of ϕ is thus equal to zero and the value of c is equal to the undrained shear strength, S_u. The values of K, n and R_f may be determined from the results of a series of tests at various confining pressures by plotting the stress-strain curves as shown in Fig. 23. By plotting the stress-strain data in the transformed manner shown on the right in the figure, it was determined that the initial tangent modulus value (E_i) was 90 kg/cm^2 and the asymptotic value of stress difference, $(\sigma_1 - \sigma_3)_{ult}$, was 0.67 kg/cm^2. The modulus number K was determined from the equation

$$E_i = K\, p_a \left(\frac{\sigma_3}{p_a}\right)^n \tag{8}$$

Because the value of confining pressure has no influence on the stress-strain curves for Atchafalaya levee clay, the value of n in equation (8) is equal to zero, and the modulus number $K = E_i/p_a$. The value of the failure ratio, R_f, is calculated from the equation

$$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}} \tag{9}$$

The values of these stress-strain parameters determined from triaxial tests on 45° specimens of Atchafalaya clay are listed in Table 9.

Using these stress-strain parameters, an incremental analysis was performed to calculate the stress-strain curve for pure shear, which Duncan and Dunlop (1969) have shown corresponds to compression with β = 45°; the calculations were performed as shown in Table 10. The initial stress conditions, before application of the horizontal shear stress, were assumed to be those which were shown in the previous section to result in correspondence between the triaxial shear strength and the shear resistance in

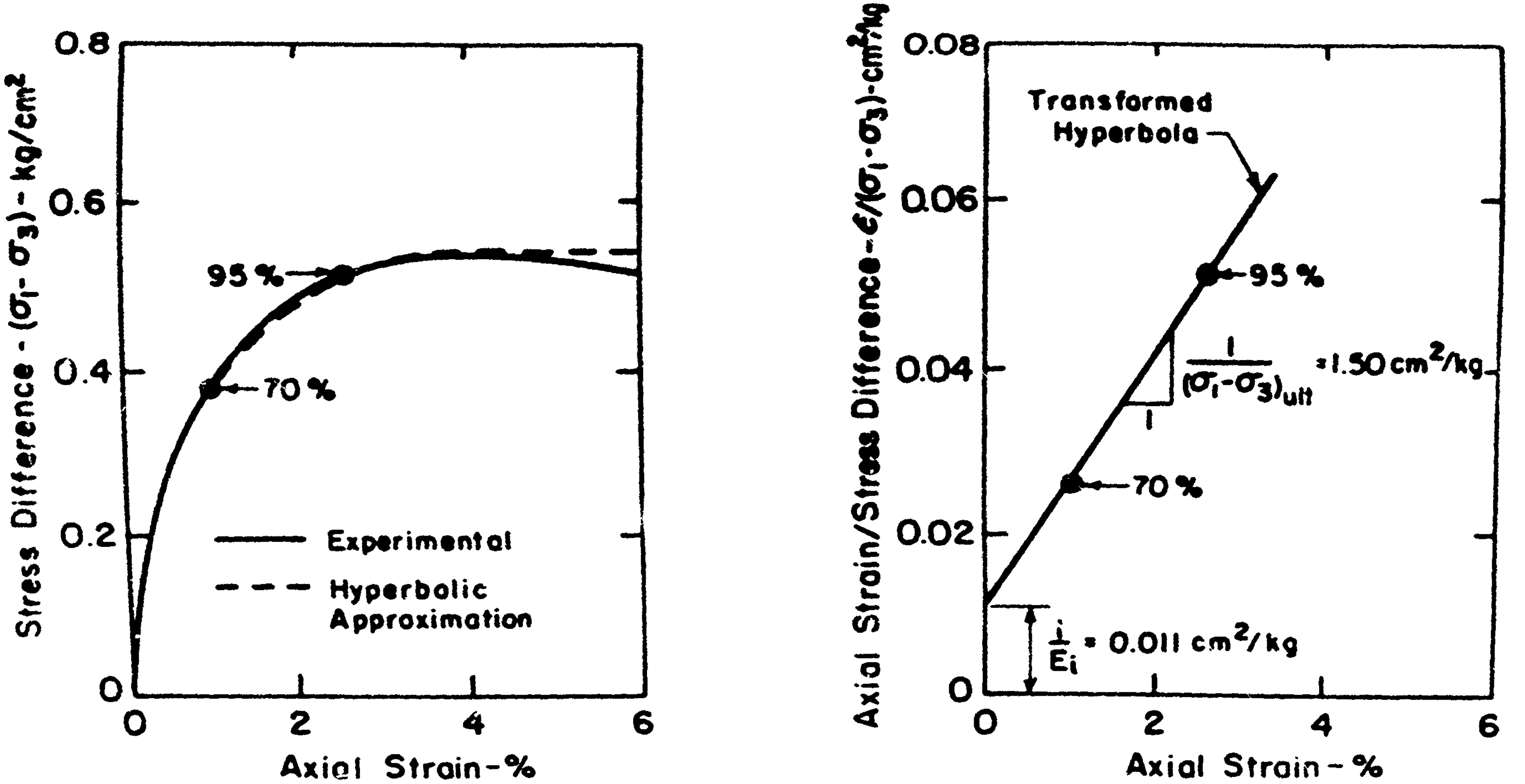


Fig. 23 ACTUAL AND TRANSFORMED STRESS-STRAIN CURVES FOR TRIAXIAL TESTS ON 45° SPECIMENS.

Table 9. VALUES OF STRESS-STRAIN PARAMETERS DETERMINED FROM TRIAXIAL TESTS ON 45° SPECIMENS OF ATCHAFALAYA CLAY

Parameter	Symbol	Value
Modulus Number	K	87
Modulus Exponent	n	0
Cohesion Intercept	c	0.27 kg/cm^2
Friction Angle	ϕ	0
Failure Ratio	R_f	0.80

Table 10. PURE SHEAR ANALYSIS

Stress Increment	Average Values for Increment			Strain Increment	Shear Stress	Shear Strain
$\Delta\tau_{xy}$ (kg/cm^2)	τ_{xy} (kg/cm^2)	τ_{max} (kg/cm^2)	E_t (kg/cm^2)	$\Delta\gamma_{xy}$ (%)	τ_{xy} (kg/cm^2)	γ_{xy} (%)
0.02	0.01	0.157	25.8	0.23	0.02	0.23
0.02	0.03	0.160	24.9	0.24	0.04	0.47
0.02	0.05	0.165	23.5	0.26	0.06	0.73
0.02	0.07	0.171	21.9	0.27	0.08	1.00
0.02	0.09	0.181	19.3	0.31	0.10	1.31
0.02	0.11	0.191	16.9	0.36	0.12	1.67
0.02	0.13	0.204	14.0	0.43	0.14	2.10
0.02	0.15	0.217	11.5	0.52	0.16	2.62
0.02	0.17	0.231	8.9	0.67	0.18	3.29
0.02	0.19	0.246	6.7	0.90	0.20	4.19
0.02	0.21	0.258	4.9	1.21	0.22	5.40

simple shear tests ($\sigma_{yo}' - \sigma_{xo}' = 0.31$ kg/cm^2). The values of tangent modulus in the fourth column were calculated using equation (7) together with the values of the parameters listed in Table 9. Using these modulus values and a value of Poisson's ratio, $\nu = 0.5$, the values of incremental shear strain were calculated from equation (6d). The resulting pure shear stress-strain curve is shown in Fig. 24, together with the average stress-strain curves from Geonor and Cambridge simple shear tests. It may be noted that the calculated pure shear curve is slightly flatter than the experimental curves, but the three variations are in fairly good agreement. The fact that the pure shear stress-strain curve is slightly flatter than either of the experimental curves may be due in part to the assumption made in the analysis that the stress conditions are completely uniform: Finite element analyses of simple shear stress conditions by Duncan and Dunlop (1969) were found to result in a slightly steeper stress-strain curve than a pure shear analysis performed using the same stress-strain parameter values.

CONCLUSION

The Atchafalaya clay tested during this study is a dark grey, highly plastic clay of medium consistency from the foundation of levee test section III. Although the clay is nonhomogeneous, containing occasional layers of silt and roots, the consistency of the results of the tests performed indicates that the clay is very uniform with regard to undrained shear strength and stress-strain behavior.

The results of unconsolidated-undrained (Q) triaxial and plane strain compression tests on vertical specimens were found to be nearly identical with regard to both undrained shear strength and stress-strain behavior.

Similarly, the results of triaxial tests performed using lubricated or normal caps and bases were essentially the same. Thus triaxial tests conducted using normal caps and bases provide a suitable means of studying many of the factors affecting the undrained strength of the clay.

The results of triaxial tests on specimens trimmed with various orientations indicates that the undrained shear strength varies somewhat with orientation of the failure plane. The values of S_u/p for the clay are quite low, varying from a maximum of 0.25 for horizontal specimens to a minimum of 0.22 for 30° specimens. The stress-strain behavior of the clay, as well as its undrained shear strength, depends on the orientations of the specimens tested. Vertical specimens were found to have the steepest stress-strain curves and horizontal specimens the flattest.

A limited series of unconsolidated-undrained (Q) creep tests were performed to study the effect of duration of loading on the undrained shear strength and stress-strain behavior. One specimen, loaded to 100% of the compressive strength measured in short-term strength tests, failed after 10 minutes. Three other specimens, loaded initially to 91%, 83%, and 74% of the undrained strength, had not failed after 20 weeks. This behavior contrasts with that found in a preliminary series of tests (Kaufman and Weaver 1967), in which a very considerable creep strength loss was observed; additional studies will be required to determine if this difference in behavior results from variations of the properties of the clay with depth beneath the surface or from other factors. Although the creep test specimens loaded to stress levels less than 100% did not fail, they continued to deform considerably. After the load had been maintained for 1 month, the strain corresponding to a stress level of 90% had increased to a value about three times as large as the strain at the same stress level in a short-

Unconsolidated-undrained (Q) simple shear tests were performed using two types of simple shear apparatus -- one manufactured by the Norwegian Geotechnical Institute (Geonor), and the other built at the University of California and patterned after a design developed at Cambridge University. The results of both types of simple shear tests were found to be essentially the same, with regard to both stress-strain behavior and shear resistance. The results of the simple shear tests were found to be consistent with the results of triaxial compression tests, provided that the initial stress conditions within the simple shear specimens and the anisotropy of the clay with respect to undrained strength and stress-strain behavior were taken into account. Because triaxial tests are easier to perform and interpret, they are believed to be more reliable and useful for determining shear strength and stress-strain parameter values for use in analyses of levee stability and movements.

The most important factors affecting the unconsolidated-undrained (Q) shear strength of the Atchafalaya clay are the effective overburden pressure (p_o') at the depth under consideration, the orientation of the specimen or direction of compression in-situ (β), and perhaps duration of loading. It would be expected that the ratio of shear strength to effective overburden pressure (S_u/p_o') measured in the tests described in this report would be representative of tests on specimens from higher and lower elevations as well, provided that the composition and stress history of the clay at these other depths is the same. Under these conditions the undrained shear strength would be expected to increase linearly with depth. The value of S_u/p_o' for the clay depends on the direction of compression and perhaps to some extent on the duration of loading, although no creep strength loss was observed in the tests conducted.

Unconsolidated-undrained (Q) simple shear tests were performed using two types of simple shear apparatus -- one manufactured by the Norwegian Geotechnical Institute (Geonor), and the other built at the University of California and patterned after a design developed at Cambridge University. The results of both types of simple shear tests were found to be essentially the same, with regard to both stress-strain behavior and shear resistance. The results of the simple shear tests were found to be consistent with the results of triaxial compression tests, provided that the initial stress conditions within the simple shear specimens and the anisotropy of the clay with respect to undrained strength and stress-strain behavior were taken into account. Because triaxial tests are easier to perform and interpret, they are believed to be more reliable and useful for determining shear strength and stress-strain parameter values for use in analyses of levee stability and movements.

The most important factors affecting the unconsolidated-undrained (Q) shear strength of the Atchafalaya clay are the effective overburden pressure (p_o') at the depth under consideration, the orientation of the specimen or direction of compression in-situ (β), and perhaps duration of loading. It would be expected that the ratio of shear strength to effective overburden pressure (S_u/p_o') measured in the tests described in this report would be representative of tests on specimens from higher and lower elevations as well, provided that the composition and stress history of the clay at these other depths is the same. Under these conditions the undrained shear strength would be expected to increase linearly with depth. The value of S_u/p_o' for the clay depends on the direction of compression and perhaps to some extent on the duration of loading, although no creep strength loss was observed in the tests conducted.

These same three factors (effective overburden pressure, direction of compression, and duration of loading) are also the most important factors affecting the stress-strain behavior of the clay. The value of the modulus number, K, like the undrained shear strength, S_u, would be expected to be proportional to the effective overburden pressure, and its value also depends on the direction of compression and the duration of loading.

ACKNOWLEDGMENTS

The laboratory tests discussed in this report were performed by Mr. R. T. Wong, member of the Soil Mechanics research staff. Gloria Pelatowsky did the drafting, and Jeanie Kwong typed the text.

LITERATURE CITED

1. Bjerrum, L. and Landva, A. (1966) "Direct Simple Shear Tests on a Norwegian Quick Clay," Geotechnique, Vol. 16, No. 1, pp. 1-20.

2. Clough, G. W. and Duncan, J. M. (1969) "Finite Element Analyses of Port Allen and Old River Locks," Report No. TE 69-3, Office of Research Services, University of California, Berkeley.

3. Duncan, J. M. and Seed, H. B. (1965a) "The Effect of Anisotropy and Reorientation of Principal Stresses on the Shear Strength of Saturated Clay," Report No. TE 65-3, Office of Research Services, University of California, Berkeley.

4. Duncan, J. M. and Seed, H. B. (1965b) "Errors in Strength Tests and Recommended Corrections,' Report No. TE 65-4, Office of Research Services, University of California, Berkeley.

5. Duncan, J. M., Seed, H. B. and Dunlop, P. (1966) "The Significance of Cap and Base Restraint in Strength Tests on Soils," Report No. TE 66-5, Office of Research Services, University of California, Berkeley.

6. Duncan, J. M. and Chang, C-Y. (1969) "Nonlinear Analyses of Stress and Strain in Soils," submitted for publication in the Journal of the Soil Mechanics and Foundations Division, ASCE.

7. Duncan, J. M. and Dunlop, P. (1969) "Behavior of Soils in Simple Shear Tests," Proceedings of The Seventh International Conference on Soil Mechanics and Foundation Engineering, Vol. I, p. 101.

8. Dunlop, P., Duncan J. M. and Seed, H. B. (1968) "Finite Element Analyses of Slopes in Soil," Report No. TE 68-3, Office of Research Services, University of California, Berkeley.

9. Kaufman, R. I. and Weaver, F. J. (1967) "Stability of Atchafalaya Levees," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM4, Proc. Paper 5312, July, 1967, pp. 157-176.

10. Kjellman, W. (1951) "Testing the Shear Strength of Clay in Sweden," Geotechnique, Vol. 2, pp. 225-232.

11. Kulhawy, F. H., Duncan, J. M. and Seed, H. B. (1969) "Finite Element Analyses of Stresses and Movements in Embankments During Construction," Report No. TE 69-4, Office of Research Services, University of California, Berkeley.

12. Roscoe, K. H. (1953) "An Apparatus for the Application of Simple Shear to Soil Samples," Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 186-191.

13. Seed, H. B., Noorany, I. and Smith, I. M. (1964) "Effects of Sampling and Disturbance on the Strength of Soft Clays," Report No. TE 64-1, Office of Research Services, University of California, Berkeley.

14. Waterways Experiment Station (1969) Memorandum regarding Second Program of Special Tests of EABPL Foundation Soils, 15 July 1969.

APPENDIX

TEST PROCEDURES

Test specimens were trimmed from the samples of Atchafalaya clay using a power band saw to remove the surrounding paraffin and to cut the samples into sections of suitable size, and using a wire saw for the final trimming operations. Water contents were routinely determined for the specimen trimmings and for the entire test specimen after testing.

Triaxial Tests

The triaxial test specimens were 1.4 in. in diameter, from 3.3 to 3.5 in. high, and were tested in the standard University of California triaxial apparatus shown in Fig. 1. The specimens were enclosed within two prophylactic membranes, each of which was 0.002 in. thick, with a layer of silicone grease between. Because the membranes were thin and the strains throughout the tests were quite small, the loads carried by the rubber membranes were negligible compared to the loads carried by the test specimens, and no membrane corrections were made. Appropriate corrections were made for piston friction.

Stress-controlled strength tests were conducted by applying the load at a rate of about 0.05 kg/cm^2 per minute initially, and about 0.025 kg/cm^2 per minute near the end of the tests. Creep specimens were loaded as rapidly as possible in the beginning of the tests; thereafter the load remained constant but the axial stress decreased a small amount as the specimens strained and their cross-sectional areas increased. Strain-controlled strength tests were conducted using a rate of 0.012 inches per minute, corresponding to a strain rate of about 0.4% per minute.

Smooth lucite caps and bases, without porous stones, were used in all tests. The lubricated caps and bases employed were larger in diameter than the specimens tested to allow for lateral expansion at the ends of the specimens. Lubrication was provided by coating the cap and base with a thin layer of silicone grease over which was placed a thin layer of rubber in contact with the specimens. Non-rotating caps were employed to prevent the specimens from sliding sideways during the tests.

Plane Strain Tests

The plane strain test specimens were 1.1 in. wide, 2.8 in. long, and 2.8 in. high. They were tested in the same type of pressure chamber as used in triaxial tests, and, as shown in Fig. 1, lucite end plates with stainless steel tie rods were used to prevent longitudinal deformation. The rubber membranes employed in the tests were 0.01 in. thick and before being stretched over the specimens were 2.0 in. in diameter. The membranes were placed on the specimens using a specially designed membrane stretcher (Duncan and Seed, 1965a). As in the case of triaxial compression tests, the required corrections for membrane loads were small and were neglected, but appropriate corrections were made for piston friction. Silicone grease was used to minimize end plate friction.

The plane strain tests were performed using a rate of 0.012 inches per minute, corresponding to a strain rate of about 0.4% per minute.

Geonor Simple Shear Tests

The Geonor simple shear specimens were 3.14 in. in diameter and 0.79 in. high. The specimens are enclosed in specially made rubber membranes (manufactured by the Norwegian Geotechnical Institute) which are reinforced by a spiral-wound steel wire embedded in the membrane to prevent lateral

expansion under action of the vertical load while at the same time allowing the specimens to deform freely in shear as shown in Fig. 1.

The Geonor apparatus is equipped with a special trimming ring which fits on the vertical motion guides during specimen preparation and provides a means of trimming the test specimen to the required diameter and height with a minimum of disturbance due to manual handling. The reinforced membrane is placed over the trimmed specimen using a specially designed membrane stretcher, and is sealed to the cap and base using stainless steel hose clamps as shown in Fig. 1.

Tests were conducted using two rates of shear strain, 1% per minute and 5% per minute, the shear strain being defined as the horizontal motion of the cap with respect to the base, divided by the specimen height.

Cambridge Simple Shear Test

The Cambridge simple shear specimens, 2.30 in. square in plan by 0.80 in. high, are enclosed in a rubber membrane and between heavy metal walls which are specially hinged so that the specimen deforms as a parallelogram under the action of the horizontal shear load. The loads carried by the rubber membrane and the loads resulting from friction between the membrane and the adjacent metal wall were quite appreciable, even though the interface was lubricated with silicone grease. The membrane and friction loads were determined by testing "specimens" of water, and appropriate corrections were applied to the measured loads as illustrated in Fig. 20.

Test specimens were trimmed to the required dimensions using a specially constructed mitre box and a wire saw. They were then placed within the 0.010 in. thick rubber membrane, which was sealed by clamping between two parts of the cap and the base. The specimens, sandwiched between the cap

expansion under action of the vertical load while at the same time allowing the specimens to deform freely in shear as shown in Fig. 1.

The Geonor apparatus is equipped with a special trimming ring which fits on the vertical motion guides during specimen preparation and provides a means of trimming the test specimen to the required diameter and height with a minimum of disturbance due to manual handling. The reinforced membrane is placed over the trimmed specimen using a specially designed membrane stretcher, and is sealed to the cap and base using stainless steel hose clamps as shown in Fig. 1.

Tests were conducted using two rates of shear strain, 1% per minute and 5% per minute, the shear strain being defined as the horizontal motion of the cap with respect to the base, divided by the specimen height.

Cambridge Simple Shear Test

The Cambridge simple shear specimens, 2.30 in. square in plan by 0.80 in. high, are enclosed in a rubber membrane and between heavy metal walls which are specially hinged so that the specimen deforms as a parallelogram under the action of the horizontal shear load. The loads carried by the rubber membrane and the loads resulting from friction between the membrane and the adjacent metal wall were quite appreciable, even though the interface was lubricated with silicone grease. The membrane and friction loads were determined by testing "specimens" of water, and appropriate corrections were applied to the measured loads as illustrated in Fig. 20.

Test specimens were trimmed to the required dimensions using a specially constructed mitre box and a wire saw. They were then placed within the 0.010 in. thick rubber membrane, which was sealed by clamping between two parts of the cap and the base. The specimens, sandwiched between the cap

and base, were then inserted in the test apparatus shown in Fig. 1. The normal load was applied through a pneumatic piston, and the shear load through a hand-operated screw jack. The tests were conducted using strain-controlled test procedures with a shear strain rate of 0.5% per minute.